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ORDINARY MEETING.

26 January, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

On the motion of the President, it was resolved :—

"That the Members present at this Meeting, on behalf of themselves and others, record the deep regret with which they have learned of the death of Sir John Audley Frederick Aspinall, D.Eng., Past-President of The Institution, and desire to express sincere sympathy with the members of his family in their bereavement."

The Council reported that they had recently transferred to the class of

Members.

Philip Piggott Brown.
John Findlay Hay.
Alfred Hartley Howarth.
George Stewart.
John Taylor Thompson, M.C.

And had admitted as

Students.

William Henry Appleyard, B.Sc. (Eng.) (*Lond.*).
Francis William Aries.
Ernest Ronald Wheeldon Aris.
Douglas Lorraine Armstrong.
Bernard William John Ashby.
Bruce Bernard Banks.
Laurence Anthony Barker, B.Sc. (*Bristol*).
John Joseph Channer Barker-Wyatt.
Kenneth Wilfred Bennett.
John Baliol Bentley.
Donald Sneath Benton.
Bertram Eric Lorimer Best-Dunkley.
Antony Francis Follett Birch.
Arthur Leonard Burcham.
Francis Stanley Burt.
Julius Cæsar, B.Sc. (Eng.) (*Lond.*).
Hugh John Campling.
Robert Wilkie Colebank.

Gordon Edison Corby.
Richard Christian Dark, B.Sc. (*Edin.*).
John Pierre Armand de Waele.
Robert Desmond Fitzgerald.
Oswald John Fox.
Ronald Tilbrook Gerrard.
Burleigh Godbolt.
Peter Anthony St. Clair Grant, B.Sc. (Eng.) (*Lond.*).
Geoffrey Ralph Herbert Griffith.
Ernest Reginald Griffiths.
John Brian Hall.
John Robson Halliday.
John Hugh Hamer.
Gustavo Donaldo Harpman, B.Sc. (Eng.) (*Lond.*).
John Alan Harvey.
Richard Haviland Haviland, B.Sc. (*Witwatersrand.*)
Arnold McNab Hawker.
Thomas Eric Ronald Hazeldine.
Frank Harrison Heaton.
John Ross Monro Hector, B.E. (*New Zealand*).
Cyril McClelland Henderson.
David McAlpine Henderson, B.Sc. (*Glas.*).
John Andrew Hilton.
Neville Alfred Hope.
Donald Powell Jackson, B.Sc., Tech. (*Manchester*).
Howard Elias Jones, B.Eng. (*Liverpool*).
Frank Ernest Ladley.
John Geoffrey Marris.
Gerald Ernest Marshall, B.Sc. (*Durham*).
John Edwin Walden Moore.
Alan Campbell Morrison.
Gubbi Naganna, B.E. (*Mysore*).
Antony Oliver Hardcastle Neilson.
Leslie Francis Offer.
William Ernest Parsons, B.Sc. (Eng.) (*Lond.*).
Geoffrey Richard Seward Paskins.
Navatkarasu Periyathamby.
Frederic Reginald George Easton Pitcher.
John Albert Posford, B.A. (*Cantab.*).
Percy Randolph Purcell.
Victor Navaratnarajah Rajaratnam.
Simon Jacob Rasanayagam.
John Clifford Rigby.
Brian Meiric Roberts.
Cyril Edwin Roberts.
Jack William Roderick, B.Sc. (*Bristol*).
Louis George Selvam.
William Joe Shobbrook.
Robert Shutt.
George James Skelt.
Arthur George Skinner.
David Hugh Hamilton Skinner.
John Percival Sowden.
William Morton Stone, B.Sc. (*Bristol*).
Thuraiappah Thiruchittampalam.
Ian Douglas Thomson.
Alexander Frederick Tindal.
Lawrence Howard Tindle.
Herbert Samuel Tricker, B.Sc. (*Birmingham*).
Norman Frederick Truscott.
Archibald Walker.
John Samuelson Walker.
William Hallam Ward.
Vincent Whitehead, B.Eng. (*Liverpool*).
Kenneth Hilton Wilkie.
Robert Dalziel Williams, B.A. (*Cantab.*).
Herbert John Melville Williamson, B.Sc. (*Cape Town*).
Herbert Alford Wilmott, B.Sc. (*Bristol*).
Ian Wilson.

The following Papers were submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 5065.

"The Salonika Plain Reclamation-Works."[1]

By Benjamin Williams Huntsman, B.Sc. (Eng.), M. Inst. C.E.

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Description of the Country.

The Plain, its rivers and their catchments.—To the west of the town of Salonika is a low-lying plain, over 500 square miles in extent (Fig. 1, Plate 1). Long ago, the sea extended to the surrounding foothills; and the Salonika Plain has been formed, in comparatively recent times, by the enormous quantities of alluvium deposited by its rivers. The converging deltas of the river Axios from the north and of the Aliakmon from the south-west closed up this former gulf of the sea, leaving behind them an inland basin of shallow water, which was gradually filled in and became the wide area of reeds and swamps known as lake Yenitsa. It has been estimated that over 250 square miles have been added to the Plain during the last 2,500 years.

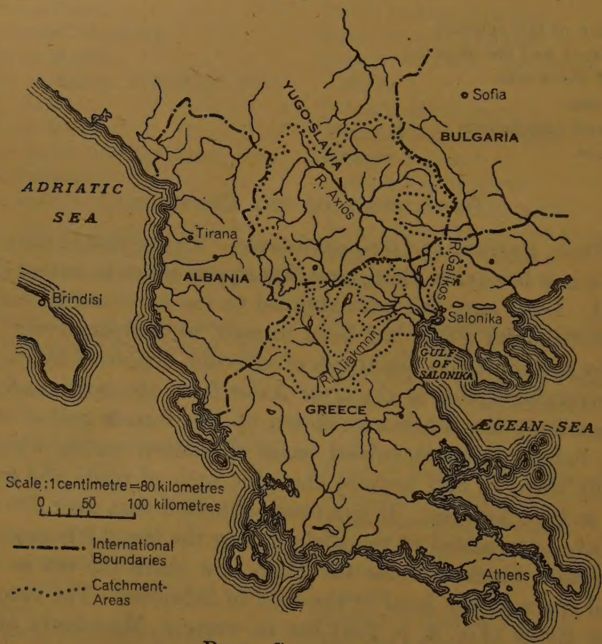
The rivers which traverse the Plain now reach the sea in a very confined space at the head of the Gulf of Salonika. The catchment of these rivers (*Fig. 2*, p. 244) lies in western Macedonia and the southern part of Yugoslavia, and is mountainous, with summits rising to between 5,000 and 8,000 feet. Rainfall is greatest in the west, and varies from about 18 inches per annum near the coast to over 35 inches in the mountains. Meteorological records are given in Appendix I (p. 277).

The rivers and torrents carry large quantities of detritus, and when they debouch from the hills their channels become too small to contain their flood-waters and their silt is spread out over the plain. Most of this silt is deposited close to the rivers' banks, and the channels of the rivers are thus raised above the surrounding land

[1] Correspondence on this Paper can be accepted until the 15th July, 1937.—Sec. Inst. C.E.

and become unstable. Floods in the larger rivers are generally due to the melting of the snow in the mountains, but heavy rainfall may cause floods at any time, particularly in the smaller catchments. Every year floods used to cause interruption of communications and destruction of property, and those cultivators who delayed sowing until after the spring floods risked the failure of their crops owing to the swift onset of the dry weather and the heat. The soil of the Plain has always been very fertile, and if protected from floods and

Fig. 2.



River-Catchments.

put under irrigation can be made very productive. Malaria is still very prevalent owing to the large areas of swamps and the lack of drainage which have hitherto existed.

The river Axios.—The river Axios, once known as the Vardar, is about 200 miles long, and about 90 per cent. of its total catchment-area of 9,200 square miles lies in Yugoslavia. After crossing the Greek frontier it flows southwards in a gorge through limestone hills until, about 36 miles from the sea, its valley opens out into lowlands north of the main Salonika Plain (Fig. 1, Plate 1). The usual flow is about 13,000 cusecs during April and about 1,000 cusecs during August. There is, however, great variation, and the minimum discharge may

fall as low as 500 cusecs. The maximum flood-discharge is estimated to be 134,000 cusecs, and the capacity of the channel is only 11 per cent. of this figure. In some places the banks of the river are as much as 10 feet above the level of the valley. Several ancient courses of the river can be traced on the east side of the plain, and early in the nineteenth century the Axios and the Loudias had a combined outlet to the sea about 13 miles from Salonika. Then, in the last years of the nineteenth century, the course of the river shifted eastwards and entered the bay within 4 miles of Salonika harbour. In the first 24 years of the present century the mouth of the Axios is estimated to have extended into the bay at the rate of about 150 feet per annum. It was estimated that the channel navigable by large ships would have become reduced to 1 mile in width by 1955, and that after that date the encroachment would have become more rapid until, in about 1975, large vessels would no longer have been able to use the port of Salonika.

The river Aliakmon.—The river Aliakmon is about 130 miles long, and has a catchment-area of about 2,700 square miles. It has a flow varying usually from about 6,000 cusecs in March to about 700 cusecs in August, although its summer flow may fall below 450 cusecs, and the largest recorded flood is estimated to have been 116,500 cusecs. The river, after debouching from its gorge, flows close to the hills at a level higher than the adjoining plain (Fig. 1, Plate 1). The normal capacity of its channel is about 13 per cent. of the greatest flood-discharge; when floods occurred the excess water used to flow towards lake Yenitsa and the Loudias, inundating large areas of land and endangering the Salonika—Monastir railway line. The Aliakmon shows more "flashy" characteristics than the Axios; floods come suddenly and do not generally last more than a day or two.

Lake Yenitsa, and the rivers draining into it.—The rivers and torrents which drained into lake Yenitsa had a total catchment-area of 1,150 square miles. The largest of these rivers, the Moglenitsa, enters the Plain from the north-west (Fig. 1, Plate 1) and has a maximum flood-discharge of over 38,000 cusecs. East of the Moglenitsa are the Balitsa, the Tchinarli and the Tchekere. West of the Moglenitsa is the Voda, which has a perennial flow, generally at least 120 cusecs, fed by springs whose waters are derived from lake Ostrovo, a large and high inland lake which has no surface outlet. South of the Voda the Arapitsa, the Kutika, and the Ana Dere drain the slopes of the Vermion mountains on the west side of the Plain.

The only outlet from lake Yenitsa was a slow-flowing narrow channel called the Loudias, and the waters of all these rivers, together

with the flood-waters of the Axios and Aliakmon which reached the lake, were trapped in this central basin and were able to drain away only very slowly. During the second half of the nineteenth century the silt brought down by the Axios blocked the Loudias, from 5 to 8 miles below the railway bridge, and increased the area of the swamps below the railway to about 34 square miles and the area of the central swamps to nearly 90 square miles. Lake Yenitsa became an almost impenetrable mass of reeds, in the midst of which was a small area of shallow open water. The bottom of the lake was soft and silty, and was less than 10 feet above the level of the sea 25 miles away.

Other areas.—East of the river Axios, and roughly parallel to it, is a depressed basin about 15 miles long, which is surrounded by low hills except for a gap, 4 miles wide, where lowlands connect it with the north end of the Axios valley (Fig. 1, Plate 1). In the bottom of this basin used to lie two lakes, Ardzan in the north and Amatovo in the south, which together formed an area of about 20 square miles of marsh, reeds, and open water. When the Axios was in flood the whole area between the river and the lakes was liable to inundation. The catchment draining into these lakes covers 280 square miles (without including the catchment of lake Doiran, which is virtually land-locked and has only a small overflow-channel connecting it with the Ayak stream which flows into lake Ardzan), and the sole outlet from lakes Amatovo and Ardzan was a circuitous and partly blocked-up drain which only functioned when the level of the lakes was abnormally high, any lowering of the water-level in the lakes during the summer being due to evaporation and absorption.

Between the town of Salonika and the valley of the Axios is the river Galikos, with a catchment-area of 400 square miles. After entering the plain it has a very shallow bed; for by far the greater part of the year it is quite dry, but floods when they came used to inundate a large area of valuable land. Many other streams and torrents enter the Plain, most of them being dry except for short periods in winter. These smaller streams have a rapid rate of rise and fall during floods, the time taken for the flow to increase to a maximum and to subside again to normal being usually not more than 24 hours.

The Contract and its Objects.

Greece is mountainous, and less than one-quarter of its area can be cultivated; although it is an agricultural country, it has to import a large percentage of its consumption of wheat and maize. The shortage of land was accentuated by the great increase in the population with the influx of refugees after 1922, and it then became

imperative to improve conditions so as to increase the yield of existing lands and to obtain further lands wherever possible. In September, 1925, a contract was signed between the Hellenic Government and the Foundation Company, of New York, whereby the Company was entrusted with the execution of a complete project and the construction of works of drainage, flood-protection, regulation of rivers, improvement of communications, and irrigation, within the area known as the Salonika Plain. This Paper deals with the design and construction of the whole of the project, with the exception of the irrigation-works.

Design of the Works.

Earlier schemes.—Messrs. Kinniple and Jaffery, between 1892 and 1896, appear to have made the first study of reclamation in the Salonika Plain. Their project proposed a diversion of the river Axios, protective embankments for this river from its gorge to the sea, a channel to drain lake Yenitsa, and embankments for the rivers entering the lake area. Neither the river Aliakmon nor lakes Amatovo and Ardzan appear to have been considered, and the whole project involved only 4,000,000 cubic yards of earthwork. A report was made in 1919 by the Société Française d'Entreprises proposing the immediate construction of a short channel to effect a partial lowering of lake Yenitsa. At a later date, a diversion of the river Axios, a left protective embankment for the river Aliakmon, and, ultimately, the completion of the drainage of lake Yenitsa, embankments for the Axios and a partial drainage of lakes Amatovo and Ardzan were to be carried out. In 1923 and 1925 other reports, Greek and German, were made. Variations of earlier schemes were proposed, and the idea of a collecting channel on the west side of the Plain was put forward.

All early investigators were agreed that the works to give protection against floods in the main rivers should take the form of earth embankments, but in the early plans many of the embankments seem to have been placed perilously close to the river banks, and in every case the floodways which were proposed were much narrower than the floodways now actually constructed. The excavation of the Loudias channel to drain lake Yenitsa was the obvious solution and was agreed upon by all, but the various proposals for dealing with the waters which flowed into the lake were by no means unanimous, and the magnitude of the problem was not, perhaps, fully realized.

Preliminary and final studies.—Preliminary surveys by the Foundation Company were commenced in November, 1925, work being started at lakes Amatovo and Ardzan, which formed a separate and independent section. The studies and designs for

this first part of the project were submitted and approved in 1926, and a preliminary study of the remaining sections was submitted in 1927; detailed surveys were then carried out and final designs prepared for each section of the works consecutively.

Estimation of "maximum" floods.—Records of floods in Macedonian rivers were very scanty. The basis for calculating the "maximum" discharge adopted for the design of the Axios works was a mark in the gorge north of the Salonika Plain showing the water-level reached during the great flood of 1907, which was the highest recorded in the last 65 years. Cross-sections of the gorge close to this point were taken, but as the exact flood-gradient of the 1907 flood was unknown it had to be estimated from such data as were available concerning other past floods. Fortunately, in 1926 another flood occurred, and from a careful observation of its surface-gradient a check could be made upon the previous calculations. A similar procedure was adopted in the case of the Aliakmon, the estimated flood-gradient being afterwards checked by measurements of floods which occurred in 1928 and 1931.

The "maximum" discharge adopted for each river was taken as the capacity for which all sections of the floodway and embankments of the river were to be designed. No allowance was made for the flattening-out of the peak of the floods due to the absorptive capacity of the berms, although in the case of the Axios, for instance, the volume of the water between the embankments and below high-flood level is equivalent to 11 hours maximum flow.

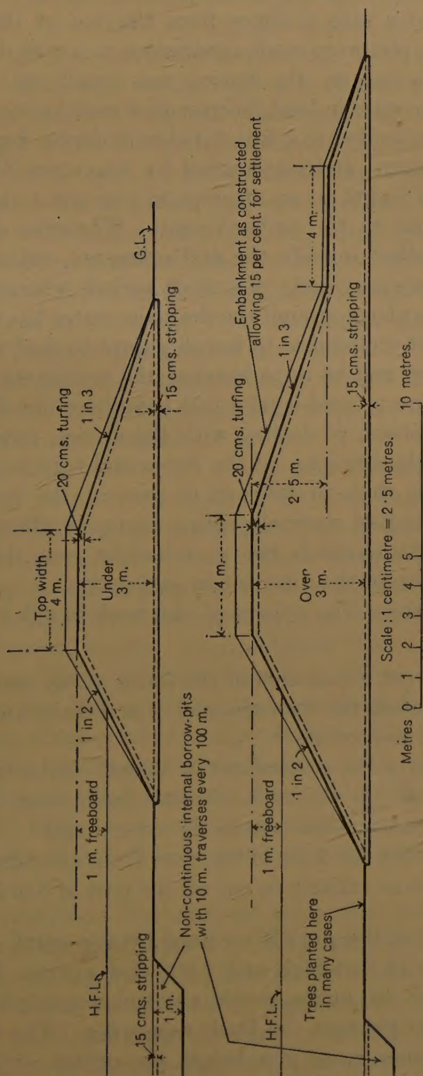
For the calculation of the discharges of the other rivers the Iszkowski formula was generally used (Appendix II, p. 278). The figures thus obtained were checked by calculations based on any available high-flood marks, and by other estimates. For the smaller rivers and torrents, the "maximum" discharges used in design were those which might be expected to occur about once in 10 years.

Drainage-channels.—Manning's formula was used in the design of river-diversion and drainage channels, the use of a more complicated formula not being warranted, as the value of the coefficient of rugosity could not be closely predetermined. For the design of channels, a carrying-capacity in cut, with water up to ground-level, equal to one-third of the maximum discharge was advocated, together with the adoption of low velocities to prevent scour. In the interests of economy, however, the Greek Government often insisted upon higher velocities and smaller amounts of excavation.

Protective embankments.—A maximum percolation-slope of 1 in 7 was adopted. This led to a standard design (*Figs. 3*) with a 13-foot crest-width, a 3-foot 3-inch freeboard, and slopes of 1 in 2 on the water-face and 1 in 3 on the back. Banquettes 13 feet wide were

added at the back, 8 feet below crest-level, whenever the embankments exceeded 10 feet in height. Soil on the site of embankments and of borrow-pits was stripped to a depth of 6 inches, and this soil

Figs. 3.



DESIGN OF PROTECTIVE EMBANKMENTS.

was afterwards placed as a top-dressing on the finished embankment, the surface being usually sown with grass-seed or planted with grass-roots. In normal material, and when constructed by hand or by

dragline, the embankments were formed with a 15-per-cent. increase in height to allow for ultimate settlement. Since borrow-pits on the river-side often encourage excessive flow and high velocities near the embankment, the safest place for them is outside the embankments at a safe distance from the toe of the slope; such external borrow-pits were made continuous to act as drains. Where the depth of water on the berms was small, or where it was essential to economize in land, borrow-pits were located on the river-side; they were limited to 3 feet 3 inches in depth, kept 16 feet clear of the embankment, and interrupted by traverses 33 feet wide in each 330 feet of length in an attempt to prevent a channel forming parallel and close to the embankment. Wherever drainage-water was liable to collect outside the embankments, culverts with non-return valves were provided. For short periods when the water-level in the channel is above ground the drainage-water has to collect outside. It was not economical to install pumps to deal with this small amount of water, and to have lowered the maximum water-level in the channels below ground-level would have been out of the question,

Silt.—The problem of dealing with the stones, sand and fine silt which are brought down by all the rivers and torrents of Macedonia was serious. In order to attempt to prevent the heavier material from entering the new channels, silting-basins or silting-reaches were provided; in the channels below, velocities were designed, where possible, to maintain the finer silt in suspension.

The river Axios.—The objects of the work in this section (Fig. 1, Plate 1) are:—

(i) to prevent inundation of the Axios valley and of the Plain;
(ii) to maintain the river-channel in as definite and permanent a path as possible;
(iii) to remove the menace to the Athens—Salonika railway line and to the Salonika—Edessa main road, and to ensure communications across the river; and
(iv) to prevent the delta of the river from extending into the sea so far as entirely to cut off the port of Salonika.

The floodway of the river is confined by two earth embankments, generally about 12 feet high and 4,600 feet apart. The berms are cleared of bushes and undergrowth, and summer cultivation of maize is encouraged to prevent the bush regrowing. The left protective embankment commences just below the outlet of the gorge and continues southwards for 11 miles, cutting off the passage of floods across the lands towards lakes Amatovo and Ardzan. It recommences from the edge of the high ground on the left bank of the river, where the valley of the Axios opens out into the main Salonika Plain,

and continues to the sea. A protective embankment on the right bank of the river above the Gorgop was not considered to be economical, and floods in the Gorgop are allowed to spread out over the land on its north bank, there depositing their silt. South of the Gorgop the Axios flows on the left side of its valley, and the centre of the valley is below the level of the river, so that drainage of the waters off the hills on the right side of the valley presented a problem. After study of various schemes it was decided to commence the right protective embankment of the river Axios at the south bank of the Gorgop, to divert the Tumba torrent northwards into the Gorgop, and to gather the waters of the other rivers and torrents into a drainage-channel, called the Vardarovassi, which follows the lowest part of the valley at a flatter gradient than that of the river until sufficient head is gained to ensure its discharge into the river, even when the river is in high flood. The cross-section of the Vardarovassi channel is as shown in Figs. 4, Plate 1. The Axios right protective embankment extends from the Gorgop for 31 miles down to the sea, unbroken except for the outlet of the Vardarovassi drainage-channel.

Floods in the past used to lose a large part of their waters over the natural spillway on the right bank of the river before they reached the existing bridges on the Salonika—Edessa main road and the Athens—Salonika railway. Both these bridges were old, and neither was big enough to pass the whole maximum discharge of the Axios. New bridges have therefore been provided, with two other road-bridges for local traffic. (Particulars of these and other bridges on the works are given in Appendix VI, p. 282.) Bridges are also provided over the Vardarovassi drainage-channel, but where the traffic is unimportant road-crossings pitched with stone are substituted, as this channel is dry for most of the year.

The Salonika—Belgrade railway line, which at one place runs close to the left bank of the river, had to be raised over a length of 2,440 yards because its rail-level was below the new high-flood level of the Axios.

The Axios diversion.—Calculations show that between 1916 and 1924 the delta of the Axios was extending into the sea at the rate of 5,900,000 cubic yards per annum, but this figure does not include the whole silt-cone which spreads out over the bay. No very accurate total could be calculated from the charts available, but an examination made by Messrs. Couper and Ker, who were consulted, showed that the average total volume brought down each year was about 20,000,000 cubic yards. This is equivalent to a denudation of the whole catchment at the high rate of 1 foot in 475 years, and to an average proportion of silt (including rolled material, which is a large

part of the whole) of about 1 in 350 throughout the year. The Foundation Company proposed a scheme involving three successive deviations of the mouth of the river. The mouth of the first diversion would have been 6 miles and that of the second about 10 miles from Salonika harbour, the idea being to fill up the available area in the bay moving southwards until, finally, the third diversion would reach a point 14 miles from Salonika. During the functioning of these successive diversions silt-deposit would have been encouraged on the land near the coast. Allowing for a navigable channel not less than 1½ mile wide and over 5 fathoms deep, it was calculated that the first diversion would have sufficed until after 1956 and the second until at least 1977; if the channel could have been reduced to 1 mile in width, these periods would have been extended by a further 15 years. The Greek Government decided, however, to proceed at once with the final diversion. The length of this diversion (Fig. 1, Plate 1) is 12 miles, about the same as that of the old channel which is now cut off. The new channel was designed to have the same capacity as that of the existing channel upstream, thus permitting summer cultivation and winter grazing on the berms, limiting the height of the protective embankments, and giving more stable conditions than could have been obtained with a channel of smaller size. Its cross-section is as shown in Figs. 4, Plate 1. The average slope from the railway to the sea is about 0·0005, but at the commencement it is about 0·00075. The new channel was designed with its bed- and water-slopes parallel to the ground. The maximum design velocities are 4 to 6½ feet per second in normal flood, and 6½ to 10 feet per second in maximum flood; on the berms the maximum velocity is limited to 4 feet per second. The ends of the embankments are bell-mouthed. At the site of the new mouth the littoral drift is very small. The action of south winds would encourage the formation of a bar, but the prevailing winds are northerly. The depth of water in the Gulf south of the new mouth is about 16 fathoms, and the river can deposit its silt here for perhaps hundreds of years; but in a much shorter time the maintenance of the adjacent mouth of the Loudias may become a difficult problem, and as the deltas of the Axios and Aliakmon extend southwards the gradients of the lower reaches of their channels will become much flatter.

Lake Yenitsa and the Loudias canal.—After preventing the flood-waters of the Axios and Aliakmon from overflowing into lake Yenitsa, there remained the problem of dealing with the waters from all the other rivers and torrents which drain to the central area. The first proposal studied was to train all these rivers into a large central basin and to dredge a large drainage-channel to the sea. However,

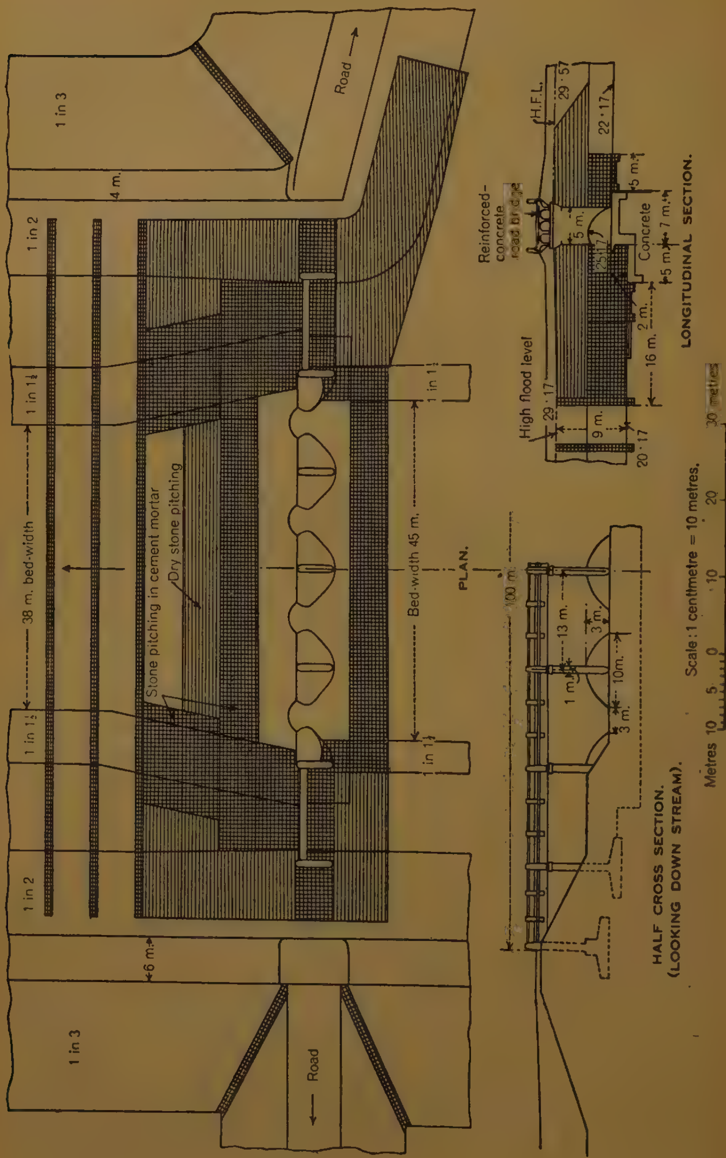
owing to the large volume of flood-water (about 60,000 cusecs), and to the flat gradient between the lake and the sea, a prohibitively large and expensive channel would have been required. The second proposal studied was to construct a circulatory collecting channel to gather the waters of all the rivers and torrents and to lead them directly or indirectly to the sea; a small drainage-channel would have been provided to take care of the rain-water falling on the area below the circulatory channel. It was found that on the west side of the Plain a circulatory channel was feasible, but that it would be uneconomical, owing to the configuration of the ground, to construct such a channel to collect the smaller streams on the north side. The northern streams were therefore trained to the centre of the lake, and from there a new channel, called the Loudias canal (Fig. 1, Plate 1), was provided to the sea. This channel is navigable to small vessels. It has a discharge of 5,300 cusecs with a depth of 24 feet 6 inches; its bed-width is 38 feet and its gradient 0·000066. Its cross-section is as shown in Figs. 4, Plate 1. The extreme maximum flood which can now enter the lake area is estimated at 12,000 cusecs; should the flood-water entering the lake exceed the capacity of the drainage-channel, a shallow inundation of the central area would occur, but the period of inundation would be short. A proposal to form a small central impounding-basin, thus limiting the area liable to flooding, was not accepted. A network of smaller drains is provided in the reclaimed land, and in the low areas it is intended ultimately to provide pumped drainage.

The Circulatory canal.—The Circulatory canal, on the west side of the Plain, commences at the Moglenitsa and has its outlet into the Aliakmon (Fig. 1, Plate 1). Its extension beyond the Moglenitsa was not found to be economical. It is located at a distance from the hills so as to collect the rivers at points where their velocities are reduced, and its alignment is arranged so as to keep the depth of cut as constant as possible, to entail the minimum amount of expropriation of villages, and to have its outlet at a suitable point in the river Aliakmon. The canal is designed to give a balance of cut and fill, and to maintain non-silting, non-scouring velocities.

The largest river, the Moglenitsa, brings down great quantities of silt, and at the head of the Circulatory canal an area of about 3 square miles is utilized to form a silt-intercepting basin for the flood-waters of the Moglenitsa and Voda. At the outlet of this basin is a structure (*Figs. 5*, p. 254) called the Upper Notched weir,[1] which provides for a drop in bed-level and converts the broad shallow flow through the basin into the deep flow in the Circulatory canal. The weir is intended to effect automatic control of the flow, so as to prevent

[1] See also *Fig. 1*, facing p. 306.—Sec. Inst. C.E.

Figs. 5.



UPPER NOTCHED WEIR.

"draw-down" with its consequent scour upstream and formation of silt-deposits downstream. Absolutely automatic control is provided for all flows up to 8,500 cusecs (about one-fifth of the maximum discharge), and it is anticipated that this figure will be exceeded only once or twice, and then only for short periods, during a normal year. When the discharge exceeds 8,500 cusecs "pooling" is already taking place in the basin, and the control, though automatic, is not perfect, the tendency being for the velocity in the upper reach to be retarded, thus aiding the deposition of silt. The design was tested with experiments on a 1/50-scale model. A bridge over the weir is incorporated in the design. With the usual swift rise and fall of floods a maximum discharge of 42,000 cusecs from the Moglenitsa and Voda will be regulated by means of the silt-intercepting basin and the upper notched weir to a maximum of 32,000 cusecs in the canal. The silting capacity of the basin is so large that a very long time must elapse before it ceases to function as a regulator and before the discharge in the Circulatory canal reaches the maximum of 42,000 cusecs for which its upper reach is designed. The canal has a maximum depth of flow of 29 feet 6 inches, a gradient of 0·0002, and a bed-width ranging from 125 feet at the head to 188 feet at the outfall, where its capacity is increased to 55,500 cusecs. The cross-section is shown in Figs. 4, Plate 1.

The flood-waters of the rivers Arapitsa, Kutika, and Ana Dere overflow their existing channels, and training embankments are provided between which part of their silt can be deposited before their waters enter the canal. Weirs are designed to lower the bed-level of the rivers to that of the canal, to check velocities, and to induce silt-deposit where silt-clearing is easy. Stone pitching is provided on the canal-embankments at these infalls to prevent erosion due to eddies and irregularity of flow. Culverts with non-return valves deal with local drainage on the west side of the canal, and small siphons under the canal maintain supplies of water used locally for agricultural and domestic purposes.

At the outlet of the Circulatory canal into the floodway of the Aliakmon, a structure called the Lower Notched weir[1] converts the deep flow in the canal into the wide and shallow flow on the river berms, and prevents "draw-down" in the canal with its attendant high velocities and consequent danger to the embankments and to the Kouloura railway-bridge a short distance upstream. There is no difference in upstream and downstream bed-levels at the weir, but in the event of the canal having a large discharge when the river-level is normal the difference in water-levels upstream and downstream may be considerable. The Lower Notched weir gives auto-

[1] See also *Fig. 2*, facing p. 306.—Sec. Inst. C.E.

matic control of the discharge with variation in the depths of flow in both the canal and the river. The two basic limiting conditions of flow assumed for the design were :—

(i) Maximum high flood in the canal (55,500 cusecs) and medium flood in the river (21,000 cusecs), with a consequent head of 14 feet 3 inches.

(ii) Summer flood in the canal (14,000 cusecs) and low flow in the river (700 cusecs), with a head of 2 feet 11 inches on the weir.

From the calculations it resulted that the total sill-width of the notch should be 76 feet; the theoretical angle which the sides of the notch should make with the vertical was so small that it was disregarded, and the weir was built with five openings each 15 feet 2½ inches wide. Each pier is 21 feet 3 inches long and nearly 16 feet in greatest width, with vertical sides, the upstream and downstream ends being tapered to give streamline flow. The piers support a road-bridge over the canal. Experiments were carried out with models to scales of 1/100 and 1/50, after which five baffles were introduced, one opposite each opening, to distribute the downstream flow more evenly.

In case floods should ever occur greater than those for which the Circulatory canal is designed, alternative schemes for overflow weirs and siphon-spillways were studied, and finally a breaching section in the embankment of the Moglenitsa basin (Fig. 1, Plate 1) was adopted. In the unexpected event of its being brought into operation it would discharge through an area of bush and shrub into the old channel of the Moglenitsa. The structure consists of a concrete floor and two abutments, with the necessary pitching, and between these abutments the top of the earthwork is at the designed high-flood level, about 3 feet below the rest of the embankment.

The river Aliakmon (Fig. 6, Plate 1).—A left protective embankment, at a safe distance from the river, commences at the foothills and continues for 24 miles down to the sea. An embankment on the right side was not considered to be economical, in view of the small amount of land to be protected. The discharge from the Circulatory canal increases the discharge of the river Aliakmon by 30 per cent. in normal conditions and by as much as 50 per cent. in flood conditions. Between the canal outlet and Milovon railway-bridge the gradient of the existing channel was 0·00036, whereas, after the closure of the embankment, the gradient of the flood-waters (which tend to follow the shortest course rather than the old meanders of the river) is 0·00071. In view of this increase in the slope, and of the addition to the flow, it was anticipated that the river would tend auto-

matically to cut off many existing loops. The project was therefore elaborated so as to suit the new conditions and to assist nature in the changes which were anticipated.

Above the outlet of the canal a diversion of the river at Sardinal was designed to cut off a long loop of the old channel. The lower end of this diversion debouches into an existing wood, consisting of large trees and dense undergrowth, with the intention of trapping the silt which it was expected would come from the scouring-out of the new channel. It was hoped that before the river had cut its own course through the wood the final natural deepening of the diversion would have been achieved. This wooded area serves also to trap debris in time of floods. A new diversion near Nishel cuts off two large northern loops of the river. Its channel, which is the same size as that of the existing channel, has a maximum velocity in flood of 8·25 feet per second. A new diversion near Alambor, about 2½ miles above Milovon, does not involve high velocities because the natural gradient around the old loop was so flat that no excessive gradient is produced by the shortened line of the diversion. Spurs and pitching on the banks are employed to train and maintain the river's course.

The existing railway bridge at Milovon was insufficient to take the increased flood-discharge, and had to be lengthened. To give sufficient waterway to limit the velocity under the bridge to 6·5 feet per second the ground at the bridge was excavated down to normal low-water level. Below Milovon the existing channel was small and its gradient was only 0·00015; as the channel was well above the level of the plain the self-transfer of the river to the left was considered to be inevitable under the new conditions. To guide this anticipated change a small "*canal d'appel*" is provided, with a gradient of 0·00044, which terminates about 4 miles below the bridge in an area of swampy land where it is intended that the river may deposit part of its silt before joining the sea.

Lakes Amatovo and Ardzan.—The winter level of the river Axios, opposite Limnotopos (Fig. 1, Plate 1), was 3 feet above the normal level of these lakes, and 17 feet above their bed at its deepest point, so that a drainage-channel could not be constructed directly from the lakes to the river. Drainage from the south end of the lakes into the river at a lower level was blocked by the dividing hills. The scheme which has been adopted provides a main drainage-channel, 8 miles long, which commences at the middle of the lake area, passes under the railway at Limnotopos, turns southward between the hills and the river, and continues parallel to the Axios left protective embankment at a flatter gradient than that of the river, until sufficient head is gained to ensure discharge. A sluice is combined with the railway

bridge at Limnotopos to control the outlet from the lakes, and a sluice at the outlet into the river prevents floods in the river from backing-up into the channel. There are drainage-channels through each lake, and small network-drains are provided in the reclaimed land. The Ayak stream, with a flood-discharge of 6,000 cusecs, enters lake Ardzan from the north, and the Selimli debouches from a gorge on the north-west side. The latter, which only flows for a few weeks each year, brings down large quantities of silt. Effective catchweirs to trap this material before its exit from the gorge would be too expensive, and these streams are therefore allowed to overflow their channels and deposit part of their detritus in silt-gathering basins before their waters are collected into the drainage-channels through the lakes. The possibility was investigated of a circulatory collecting channel around the north side of lake Ardzan discharging directly into the Axios, but as such a channel would have had to commence below the silt-gathering basins of the Ayak and the Selimli it would have been at too low a level to have been able to discharge into the river. It was not possible to give the main drainage-channel a discharge-capacity equal to the maximum inflow into the lakes, and consequently a certain area in the lowest part is liable to temporary inundation during heavy floods.

The river Galikos.—Protective embankments have been provided, at a distance of about 2,600 feet apart, from the edge of the foothills down to a point 2 miles below the Athens—Salonika railway. Below this point the flood-waters spread out in an area of waste land, not yet worth protecting, where their silt can be deposited before the waters drain into the Bay. The railway bridges were of sufficient length to pass the flood-discharge, but it has been necessary to pitch the bed of the channel at each bridge so as to maintain a limited depth of flow and restrict the velocity.

Other works.—Twenty-nine bridges (Appendix VI, p. 282), having an aggregate length of 12,700 feet, were designed and constructed, as as well other smaller bridges and culverts. Six railway bridges and four road-bridges consist of steel spans on concrete piers and abutments founded, with one exception, on piles. Eight road-bridges are of reinforced concrete. Six bridges for minor roads are constructed of timber, and two have timber decking supported by reinforced-concrete piles and columns. One temporary railway bridge consisted of steel girders on timber piers, and one temporary road-bridge was designed with removable pontoons and timber trestle approaches. New roads were built across river berms and for the approaches to bridges, and a number of service roads were necessary.

Weirs in the new channels were generally constructed of masonry with timber sheet-piling, but concrete and steel sheet-piling were

used for larger structures, and stones in wire-netting boxes in conjunction with timber piling were tried for smaller channels. Either dry stone pitching or stone pitching in cement mortar was employed for the protective revetment of embankments, because local masonry is cheap and usually of good quality. Wire-netting bags filled with stones were used for the revetment of the banks of rivers and for spurs, but spurs formed of stone-filled triangular timber frames with brushwood on top were also tried. The maintenance and control of the rivers' channels is a never-ending undertaking, and every summer some new training works have to be constructed.

Construction.

The Axios works.—The construction of the upper left protective embankment of the Axios (Fig. 1, Plate 1), was undertaken at the same time as the works of lakes Amatovo and Ardzan. The first 10 miles of the Axios right protective embankment, below the Gorgop, were completed in 1929. In that year the main works in the Plain were commenced; but below the already completed section of this right embankment a gap, $4\frac{1}{2}$ miles long, had to be left open for some time longer so that floods could continue to escape over the natural spillway on the right bank. The reasons for this were threefold. Firstly, it was necessary to complete the new railway and road bridges, together with the widenings of the channels under these bridges, before the maximum flood-discharge could be passed under them. Secondly, until the new diversion-channel had been completed, the river below the railway had to follow its old course to the sea, and any increase in the volume of flood-water in that course would have resulted in the inundation of several villages and a large area of cultivated land. Thirdly, the closure of the embankments above the railway would have resulted, during a flood, in a confined and deep flow of water which, after passing the railway bridge, would have spread out suddenly to a shallow depth over a wide area, inducing dangerously high velocities under the bridge.

The new bridges and their river-widenings were completed by 1933, and the completion of the excavation of the Axios diversion was scheduled for the autumn of that year. Delay in this work, however, necessitated the diversion remaining unopen longer than anticipated, and a flood actually occurred, shortly after the Axios right embankment had been completed and just before the diversion could be opened; its effects were, luckily, not very serious. The completion of the Vardarovassi channel, which crosses the former natural spillway of the Axios, had also to be postponed until after the closure of the Axios right embankment.

The Aliakmon works.—The programme for construction had to be arranged so that the floods which might occur during the seasons 1931–32 and 1932–33, while these works were in progress, should cause the least possible damage to the semi-completed work, and so that the construction itself should not aggravate flood-conditions or increase the existing danger to neighbouring villages. Until Milovon railway-bridge had been extended and the new widening of the river-channel completed, the existing flood-escapes on the left bank of the river had to remain open. The alignment of the Aliakmon left embankment (Fig. 6, Plate 1) crossed several old loops of the river, and the closures of this embankment at these crossings could not be made until the new diversion-channels had been excavated and the new road-bridge at Nishel completed. Owing to the large amount of work which had necessarily to be postponed until after the spring of 1933 and then hastily completed during the summer and autumn, and also on account of a shortage of labour, much anxiety was felt lest an early flood should have come before the embankment had been closed. The closures were, however, safely effected before the flood-season of 1933–34, and minor works and pitching were continued during 1934 after the embankment had become more consolidated.

The works of the Circulatory canal.—No extra water could have been added to the Aliakmon until late in 1933, so that the Circulatory canal could not have been completed before then. In fact, owing to the large quantity of earthwork involved in this section, completion was scheduled for the autumn of 1934. Provision had to be made for allowing the flood-waters of the Moglenitsa, Voda, Arapitsa, Kutika, and Ana Dere to flow into lake Yenitsa, as in the past, until 1934; for this purpose sections of the canal were left unexcavated, and a gap was left open in the Moglenitsa embankment. Excavation of the canal was commenced in sections between each river, each section being kept dry during construction by temporary drainage-channels. The construction of the infalls of each river into the canal had to be so planned as not to block the waterway of these rivers while they continued to flow into lake Yenitsa. A flood-escape, about 2 miles wide, had to be left open for the Aliakmon floods to cross the alignment of the Circulatory canal north-west of Kouloura, and this section of the canal could not be commenced until the summer of 1933. All these considerations complicated the work, and necessitated the postponement of many items of construction until the last season, when everything had to be rushed to completion as quickly as possible. The closures were effected late in 1934, and during 1935 all the remaining minor works were completed.

The works of lake Yenitsa.—Floating plant was used to excavate the channels in the lake (Fig. 1, Plate 1), and the water-level had to be

maintained while this was being done. As the ground would have been too soft for the employment of land-machines until a considerable time after the lake had been drained, the excavation of channels was carried out by floating plant as far upstream as flotation would permit while the water-level was high. A dam was necessary to maintain this water-level. The dam had to be capable of passing maximum floods with safety and without raising the water-level so high as to cause extra inundation of surrounding lands. The proposal of a temporary overflow-weir in the channel just below the railway-bridge was abandoned in favour of a non-overflow earth dam $7\frac{1}{2}$ miles further downstream. The soil here was suitable, and the site enabled a greater length of channel to be excavated without supplies and transport to the machines having to cross the dam. Floods could escape over a long natural spillway on the right bank of the channel above the dam.

The excavation of the channels in the lake had to commence from the depot at the railway bridge, the machines first making a cut just sufficient for their own flotation; but as a general rule excavation proceeded in the downstream direction so that silt, caused by the underwater excavation and carried downstream by the current, could be cleared up as the machine moved forward.

Flood-waters from the Axios and the Aliakmon were liable to overflow into the lake until 1933, and all the waters off the hills on the west side of the Plain would continue to reach the lake until after the closure of the Circulatory canal late in 1934. The removal of the dam and the lowering of the water-level was not desirable while a large inflow of water into the lake remained possible, because scouring of the banks and deposit of silt in the channels would have resulted; on the other hand, an early lowering of the water-level was demanded because more land around the edge of the lake was urgently required for cultivation. Actually, the dam was cut and drainage of the lake commenced on the 15th June, 1933.

Earthworks by hand-labour and by machine.—A total quantity of 61 million cubic yards of earth was moved. Of this, 19 per cent. was done by hand-labour and 81 per cent. by machines. The maximum output in 1 year was 16,860,000 cubic yards, in 1933; and the maximum output in 1 month was 2,185,000 cubic yards, in September, 1933. Labour-rates were low, and earthworks could be done cheaply by hand-labour, using local carts to transport the spoil from the borrow-pits to the embankments. During the periods of ploughing and of harvesting, however, labour was difficult to obtain even when high wages were offered; and this difficulty increased each year as more and more land was put under the plough.

For the excavation of channels and for the formation of embank-

ments by machine, dragline excavators, mounted on caterpillar-tracks, were generally used; a small amount of work was also done by tractors and scrapers, and one large multi-bucket excavator was employed on part of the Circulatory canal. For underwater excavation two large suction-cutter dredgers and four dragline excavators mounted on pontoons were used. A total of 29,300,000 cubic yards was handled by land-machines, and 20,200,000 cubic yards were excavated by floating equipment. Particulars of the excavating plant employed, together with details of outputs, fuel-consumption and running-costs are given in Appendixes III, IV, and V, pp. 279–81.

The first machines to be purchased were three 2½-cubic-yard coal-burning, steam-driven draglines. Their work commenced in 1927, and by 1935 they had excavated over 8,000,000 cubic yards. They proved to be very satisfactory and reliable, but their operating-costs per cubic yard were about 75 per cent. higher than those of diesel-powered machines.

The channels in lakes Amatovo and Ardzan were excavated by two 1¾-cubic-yard draglines, with 70-foot booms, mounted on pontoons. Longer booms, giving a greater dumping-reach, might have been more suitable, as some trouble was experienced with soft spoil slipping back into the cut. After the completion of their work in lakes Amatovo and Ardzan these machines were transferred to work in lake Yenitsa.

Two diesel-electric draglines, mounted on pontoons, with 5-cubic-yard buckets and 125-foot booms (or, with 3½-cubic-yard buckets and 137-foot booms) were ordered in 1929. When these machines commenced work in 1930, trouble was experienced, as the large buckets proved to be too great a load; later, after adjustments had been made, they gave very good outputs with smaller buckets. They were first employed in opening-up the top cut of the Loudias canal, in order to remove the reed-roots and leave a clean face for the suction-cutter dredger which was to complete the section. The spoil dumped by the draglines served to form bunds behind which the dredger's effluent could be deposited (Figs. 4, Plate 1). The machines were afterwards used in excavating channels to full section. A slightly lower output resulted when excavation had to be done in a great depth of water. Clam-shell buckets gave better results than drag buckets, particularly when working into a face was unavoidable.

A 21·6-inch diesel-electric suction-cutter dredger, with a maximum dredging-depth of 26 feet, was ordered for work in the Loudias canal. Spoil was discharged to either side through long shoot pipes carried by a framework mounted on a terminal pontoon connected to the dredger by a length of 200 feet of floating pipe-line. Water-ballast was used on the opposite side of the pontoon to that to which

the spoil was being discharged. The out-reach of the discharge-pipe to either side was 180 feet from the centre of the pontoon. The spud-gear on the dredger was of the rotary type. The dredger was built in Venice and was towed from there to Salonika, arriving in September, 1930. As it was about to enter the Loudias canal from the sea a strong southerly gale suddenly sprang up, the moorings were torn away, and one of the spuds was broken. The dredger was brought back to Salonika harbour for repairs. One month later it was taken out again and, this time, safely towed up the existing Loudias channel, locked through the dam, and, after undergoing trials, commenced work in the upper part of the canal.

Six diesel-engined draglines, of various sizes from $\frac{3}{4}$ cubic yard to 2 cubic yards, were employed on channel-excavation and on embankment-formation, and also one petrol-engined dragline which was later converted to diesel power. All these small draglines proved to be fast and satisfactory machines, and with diesel engines their operating costs were very low. In the widenings of the river-channels, where spoil had to be transported over distances up to 700 yards, 7-cubic-yard dump-waggons were used drawn by 60-HP. tractors.

Special plant had to be chosen for the Circulatory canal where the maximum top width of the channel was 225 feet and the embankments were 400 feet apart (Figs. 4, Plate 1). Two 6-cubic-yard draglines, with booms 155 feet long, and one large multi-bucket excavator were employed on this work, all being diesel-electric. The draglines were each mounted on eight caterpillar tracks ; they were heavy, and had to stand on timber mats in wet weather. Certain mechanical defects delayed their work at the commencement, but after these had been remedied the draglines proved very satisfactory. The multi-bucket excavator was ordered in 1930 and commenced work in May, 1931. It was mounted on rails, with a separate engine-house, had twenty-seven buckets each of 14 cubic feet, and a bucket-arm adjustable to the size of cut to be excavated. Its main conveyor-belt was 130 feet long, and could be slued through 180 degrees so as to dump the spoil where required. During its tests an output of 537 cubic yards per hour was obtained in ordinary soil, and 311 cubic yards per hour in a deep cut of gravelly earth with about 30 inches of water at the bottom. The machine was capable of large outputs in straight cuts in normal ground, but wherever the work was not perfectly straightforward other plant had to be employed. Owing to the manner in which the multi-bucket machine deposited its spoil lightly and finely in the embankments, these had to be formed with 20 per cent. allowance in height for settlement, and the embankments so formed took much longer to consolidate and were not so strong as those formed by draglines.

A suction-cutter dredger was used to excavate the Axios diversion-channel. In order to have flotation, the dredger had to commence at the top, water being supplied through a sluice from the existing river. It was originally intended to pump the spoil from the channel to form the protective embankments, and schemes were elaborated for a pipe-line that could have been moved forward without being dismantled; pumping would have taken place to each side alternately, and the spoil would have been deposited between small previously-formed retaining bunds. In the interests of economy and speed of execution, however, the Government decided that spoil from the channel was to be deposited on the berms close to the edge of the channel, the protective embankments being formed from adjacent borrow-pits (Figs. 4, Plate 1). This scheme had the objection that it would prevent free interchange of water between the central channel and the two berms in time of flood, and would virtually divide the flow into three parts; it was feared that, owing to the levels of the ground on the berms, the flood-water-levels on the berms and in the channel would not remain the same throughout the length of the diversion, and that in some places the water on the berms would rise above that in the central channel. Where this would occur, and when the water would rise above the level of the spoil-heaps, it was feared that it would pour into the central channel, carrying with it that material, which would be deposited in the channel farther downstream. The dredger which was ordered for this work was a 25·6-inch suction-cutter dredger driven by two four-stroke diesel engines, each of 900 HP., coupled to generators. Two separate pumps and pump-motors were installed, so that no delay in working should occur due to breakdown, or changing of impellors or liner-plates; each pump-motor was of 1,100 HP. and could be supplied with current from either or both of the two generators. The cutter-motor was of 200 HP., the spud-gear was of the rotary type, all controls were worked from the operating-house, and accommodation was provided on board for all the crew. To reach the top of the diversion it was necessary to bring the dredger up the existing river. The dredger left the works in Holland in March, 1932, and arrived at Salonika 5 weeks later. After excavating through the bar at the mouth, it entered the deeper water of the river and commenced moving upstream. Owing to delay in the placing of the order, the passage up the river commenced later than originally intended; unfortunately in that year the water-level dropped very rapidly, so that flotation became difficult, some dredging was required, and progress was delayed. The discharge in the river that season proved to be the lowest hitherto experienced, and the water-levels

were so exceptionally low that it was finally decided to stop the dredger and to await the rise of the river later in the year. After it had been decided to deposit the spoil close to the sides of the channel, a terminal pontoon with an overhanging discharge-pipe was designed and constructed locally, and small bunds about 5 feet high were formed on either side of the channel to retain the spoil. During the trials of the dredger an output of 1,130 cubic yards per hour was obtained with 1,173 HP. developed by the engines; the velocity in the pipe-line was about 15½ feet per second, with 15 per cent. of solids, and the pump-efficiency was 54 per cent. With only one engine working an output of 805 cubic yards per hour was obtained. The dredger commenced the excavation of the diversion-channel in April, 1933, and completed 4,667,000 cubic yards in 11 months.

Organization.—The Company's head office for the contract was in Salonika, and work in the field was organized in three divisions, well-built quarters being constructed at a suitable site on each division. The quarters for No. 1 division were at Amatovo; No. 2 division included the works of the Axios and of Yenitsa, with its quarters at Nea Halkedon, and quarters for No. 3 division, which comprised the works of the Aliakmon and the Circulatory canal, were at Ergohori, near Verria. Main depots with machine-shops, stores and dwelling huts were built on each division, and smaller depots and stores were set up where necessary according to the needs of the work. Owing to the generally bad state of the few roads which existed and to the long distances which had to be travelled on land and by water, the cost of transport was high and the supply of fuel to machines was often difficult.

The construction of the larger bridges, whose piers and abutments were generally of mass concrete founded on timber piles, involved the excavation of over seventy cofferdams. As many of these bridges could be built before the channels under them were excavated the greater number of these cofferdams were excavated from dry land. In 55 months eleven major structures were completed, involving the driving of 240,000 linear feet of timber piles and the placing of over 45,000 cubic yards of concrete.

Owing to the isolated positions of many sections of the works and to the time taken in going to and fro, work was carried on continuously except for breaks of 2 or 3 days once a fortnight; this enabled better outputs to be obtained than by stopping for one or two days every week. Medical charges and payments which had to be made for compensation for sickness and accidents were heavy. As a general rule plant was not insured, the Government accepting the risk of damage or loss. The Company was required

to recruit its technical assistant and labour personnel as far as possible among Greek subjects, and foreign assistant engineers, foremen and machine-operators were replaced as soon as Greeks could be trained to take their places. Local operators proved to be efficient if well supervised, but a longer time was necessary for the satisfactory replacement of foreign supervision and of those responsible for the maintenance of the machines. The number of men employed rose at one period to 4,300 and averaged about 2,800.

Results and Conclusions.

Floods.—Since the completion of the works exceptionally large floods have occurred in several rivers; it is interesting to compare them with the "maximum" discharges for which the works were designed. The largest floods in the Axios during the present century occurred in 1907, 1923, and 1935, and large floods also occurred during the construction of the works in February and April, 1931, and in January, 1934. Between 1926 and 1935 the average value of the greatest discharge each year has been about 30 per cent. of the assumed "maximum" value. The duration and height of the floods varied considerably in different seasons. In 1931, for example, flooding lasted for a total of 65 days, whereas in 1933 the discharge of the river at no time exceeded 10 per cent. of the "maximum." The flood in the Axios in 1907 (which was the highest recorded, and which is believed to have been the highest which actually occurred, in the last 65 years) was estimated to have been 123,000 cusecs, and the flood which reached its peak on the 11th December, 1935, was estimated at 86,200 cusecs, whereas the Axios works were designed for 134,000 cusecs.

In December, 1935, heavy rainfall, which was particularly heavy in the catchments on the west of the Plain, caused exceptional floods in many rivers. The highest water-levels of these floods were recorded and estimates were made as carefully as possible of the maximum discharges which occurred. Appendix VII, p. 284, shows the estimated discharges at the peak of various floods, together with the corresponding rates of run-off. The peak of the Aliakmon flood in December, 1935, far exceeded all floods known in the past. The discharge in the gorge, just before the river entered the Plain, was estimated at 116,500 cusecs, and at Milovon the estimated discharge was 142,600 cusecs, whereas the "maximum" which had been adopted in the design of the protective works was only 121,700 cusecs. The flood of December, 1935, in the Axios lasted 7 days; that in the Aliakmon lasted 4 days, and exceeded the design "maximum" for a period of about 12 hours in the gorge and about

6 hours at Milovon. In the Ana Dere the flood-discharge, calculated from careful measurements made directly after the flood had occurred, was 22,400 cusecs. This is equivalent to a run-off of 350 cusecs per square mile. At a station at the foot of the hills and at the edge of this catchment over 4·3 inches of rain in 13 hours was measured during the night of the 9th–10th December, 1935, the heaviest rainfall during this period lasting about 4 hours. In the gorge through the town of Verria this flood rose so high that it washed out an ancient bridge which is reputed to have been standing for over 300 years, and no memory could recall nor records show any previous flood comparable with it. Part of the flood-waters of the Ana Dere escaped to either side of its channel before reaching the infall into the Circulatory canal; other streams and torrents off these hills also had excessive floods, and a pool of water formed on the west side of the Circulatory canal, inundating about 3,000 acres and flooding one or two villages. The water in the canal dropped below berm-level by the 11th December, and all this water outside the right embankment drained into the canal by the 13th December. Owing to the excessive discharge in the Aliakmon, its left protective embankment was topped and breached at several points above the outfall of the Circulatory canal. Below the outfall of the Circulatory canal no damage occurred at any point of the Aliakmon left embankment, although at one place the water rose 3 feet 6 inches above the anticipated high-flood level, and was only prevented from topping the embankment by the use of sandbags. In the whole of the Plain east of the circulatory canal and north of the Aliakmon no flooding outside the embankments occurred during the floods of December, 1935. In view of the discharge in the Aliakmon having exceeded the assumed value, the strengthening and raising of its left protective embankment has been put in hand.

Embankments.—Some trouble was experienced during the first two seasons after the formation of the embankments, and the action of flood-water on the comparatively "green" embankments caused erosion and slipping of the inner face in some places, particularly in localities where the soil was lighter or sandier than normal. Wind was often the determining factor: a strong wind might cause wave-action which would quickly erode the face of an embankment, whereas equally high water on a calm day would do little or no damage. The cross-section of embankments which had been adopted proved to be generally satisfactory, and a few breaches which did occur were mostly due to water topping the embankments. Seepage through the embankments occurred only in a few instances. Some trouble occurred with moles, which came up into the embankments at the beginning of each flood-season when the ground on the

berms began to get wet, and attempts were made to drive them out, to catch them or to poison them.

A sufficiently good sod of grass was obtained in some places simply by replacing the top stripping on the embankment after its formation. In 1931 and 1932 some embankments were sown with a special mixture of seed obtained from Italy, about 50 pounds per acre being used. Previous to this sowing the ground was treated with about 500 pounds of chemical manure per acre. Sowing was done in the spring and autumn at times when rainfall was likely; no sowing could be done during the heat of the summer or in the cold weather during the winter. This grass gave very good results, but was expensive. Later, couch grass roots, collected locally, were planted on many of the embankments. This was much cheaper (costing about £2 per acre), but was not so satisfactory, and in many cases the roots had not time in which to spread before the embankments were subjected to the action of floods. In cases where temporary protection was required quickly, barley was sown.

Over fifty watch-huts were established on the embankments, at intervals of about 3 miles, with one permanent watchman always at each post. From 1st December to 30th April during each flood-season two extra men were appointed to each post. During normal times these extra men were formed into maintenance-gangs, but at periods when the rivers were high or when, for any reason, floods might be expected, these men carried out the duties of extra watchmen, so that watching could then be continuous throughout the 24 hours. A telephone system is being constructed so that most of the watch-posts will be in direct communication with headquarters.

New channels.—Excavation with 1-in-1½ side-slopes proved suitable in normal ground. In the Circulatory canal, where the soil is firm and where the depth of cut is about 15 feet, these slopes have satisfactorily maintained their shape during the first two seasons. In the reclaimed land the network-drains were excavated with 1-in-1 side-slopes (these steeper slopes being considered more effective in draining the ground-water), and here a certain amount of slipping occurred during wet weather. In underwater dredging the cross-sections were not excavated exactly in accordance with design, nor would a greater expenditure have been justified in attempting to produce sides at exactly the theoretically correct slope.

Excavation by floating plant generally necessitated the removal of a greater amount of material than the estimated net quantity. Sometimes part of the soft material excavated slipped back into the cut; in some cases the inflow of water into the cut during and after construction brought in silt which was deposited in the channel,

and in other cases slips occurred after the slopes of the cut had been excavated. In the channels of lakes Amatovo and Ardzan the total quantity excavated exceeded the estimate by 25 per cent., and in the main drain of lake Yenitsa 15½ per cent. extra had to be excavated. In the combined channel of the Tchekere and Tchinarli 29½ per cent. extra had to be excavated, but most of this was due to silt brought down, during construction, from the channels upstream. In the Loudias canal the measured excavation exceeded the estimate by 3½ per cent.

At the mouth of the Loudias a temporary channel had been excavated in March, 1931, to a depth of about 10 feet below sea-level. This channel later became partly filled-up, and was re-excavated in March, 1934. Only a small amount of silt is carried by the Loudias, and very little silt-deposit occurs near its mouth, but the action of wind and waves caused a small bar to form which had risen to about 5 feet below sea-level by April, 1935.

Observations made in the Loudias canal in January, 1935, showed that the average time of high water at the railway bridge, 13 miles from the sea, was a little over 1 hour later than the time of high tide in the sea; also that high water at the head of the canal, 25 miles from the sea, occurred about 2 hours after high tide in the sea. The range of the water-levels was 11·8 inches in Salonika bay, 5·95 inches at the railway bridge, and 4·4 inches at the head of the canal.

The Axios diversion-channel.—The river was diverted down the new channel on the 5th April, 1934. The flow decreased from about 6,000 cusecs in April to less than 800 in August, and there was no appreciable rise until October, 1934. During the first flood-season there were two or three high-water periods, during which the discharge did not exceed about 32,000 cusecs, and at the beginning of the second flood-season, in December, 1935, a larger flood occurred, as mentioned above. The general tendency of the river after diversion was to scour its bed in the upper reaches and to deposit silt in the lower reaches. In the top reach for a length of about 3 miles, 275,000 cubic yards was scoured out of the bed in the first 3 weeks, resulting in the lowering of the bed by nearly 2 feet. The volume of earth thus scoured out was equivalent to about 1 part in 1,200 of the water passing during that period. During the next 12 weeks a further 220,000 cubic yards was removed, equivalent to about 1 part in 4,200 of the water passing. During the summer of 1934 there was not very much erosion of the bed of this top reach, but some erosion of the banks occurred. In the first flood-season scouring of the bed recommenced, and at one point the top-width of the channel was increased from 275 feet to 350 feet by December,

1934, and to 480 feet by March, 1935. The mean velocity in this top reach on the 17th April, 1934, was 4·75 feet per second. This velocity had decreased to 2·80 feet per second by the middle of June and to 1·90 foot per second by the middle of July. The greatest bottom-velocity (12 inches above the bed) observed was 5·375 feet per second in April. Initially the water-surface in this reach had a slope of 0·00078, but 8 weeks after opening it had flattened to 0·00075, and 1 year later, at approximately the same discharge, the water-slope was about 0·00056. Velocities during the first 3 weeks of flow in the new channel were about 40 per cent. higher than the velocities in the existing river upstream. The scouring in this top reach, which was to some extent to have been expected while the river was assuming a new regime, was increased by the high initial velocities and by the very light and friable nature of the soil. The river, in deepening the top reach of its diversion, also deepened its existing channel as far as 8 miles above the commencement of the new diversion-channel. The extent of this deepening was not foreseen when the diversion was designed. It had the good effect of increasing the discharge-capacity of the river's channel; but the deepening of the bed under the Axios railway-bridge necessitated the dumping of stone as protection around some of the piers. A little scouring occurred during the large flood of December, 1935, but as the gradient of the river has now become more uniform throughout its length it is not anticipated that serious deepening of the bed in the top reach of the diversion will recur.

A little silt was deposited in the middle reach of the diversion (which has a gradient of 0·0005) in April, 1934, but practically all the material scoured out of the top reach was transported past the middle reach without deposition. In the lowest reach of the diversion the initial velocities were less than those in the upper reaches. By the end of the first 2 weeks a wedge of silt, with its thick end downstream, and about 3½ miles long, had formed in the lower reaches. This wedge was being rolled along the bottom of the channel by the current, which increased in velocity to about 6·2 feet per second at this point 2 weeks after opening. During the second half of April the front of the silt-wedge moved forward about ¾ mile, at an average speed of 13 feet per hour. During May the front of the wedge continued to advance, and scouring of parts of the wedge occurred. By the 1st June the average velocity in this part of the lowest reach had dropped to 2·9 feet per second, and thereafter the transport of the silt became much slower.

The dredger finished the excavation of the diversion on the 26th March, 1934, and between that date and the opening of the diversion on the 5th April rough seas deposited about 46,000 cubic

yards of sand in the mouth of the channel. When the diversion was opened, 33,000 cubic yards of this sand were scoured out in the first six days. The velocity at the mouth reached 7·9 feet per second during April, 1934, fell to 6·1 feet per second by the end of that month, and at the beginning of June was only 2·3 feet per second. A shallow bar began to form at the mouth of the diversion in May, 1934. Since April, 1934, the mouth of the diversion has extended into the sea in a straight line, the silt-banks which have formed on either side being a uniform distance apart except for a slight bell-mouth at their extremities. The depth of water over the bar is normally about 2 or 3 feet. The following figures show the advance of the bar at the mouth of the river southwards from the original shore in April, 1934.

Month.	June, 1934.	November, 1934.	January, 1936.
Total advance . . .	570 yards.	1,090 yards.	1,290 yards.
Total period . . .	2 months.	7 months.	21 months.

The river Aliakmon.—The works which were executed between 1931 and 1934 (Fig. 6, Plate 1) cut off five loops of the river, having a total length of 33,200 yards, by diversion-channels having a total length of only 12,300 yards, and three other loops cut off themselves automatically. As the result of all these cuts-off, natural and artificial, the course of the river's channel from the commencement of the left protective embankment down to Milovon bridge was shortened from 34·3 miles to 18·9 miles. From the commencement of the left protective embankment down to the outlet of the Circulatory canal the average gradient of the river's bed was increased from 0·00066 to 0·00120, and from the outlet of the Circulatory canal down to Milovon bridge it was increased from 0·00036 to about 0·00065. The river from Milovon to the sea was shortened from 13 miles to about 7 miles. After the first two flood-seasons it was found that the bed had been lowered by scouring in the upper reaches and raised by silting in the lower reaches. The average gradient of the bed was thus reduced from 0·00120 to about 0·00105 in the upper reaches, and from 0·00065 to about 0·000575 below the outlet of the Circulatory canal. The new diversions of the river suffered no bank-erosion during the first three seasons, with the exception of some local scouring where the soil was very light and friable. The cuts-off which occurred automatically have not been so satisfactory. When the river effects such a cut-off it does not form a well-defined new channel, and its new bed is generally irregular in alignment and insufficient in discharge-capacity; experience shows that in such cases meandering and bank-erosion soon recommence unless extensive training works are undertaken. By the end of the second flood-season the river

had cut for itself a defined channel through the wooded area below the Sardinal diversion, but in the meantime this area had served its main purpose, and the material scoured out in the reaches of the river above it had been deposited there without much silt being carried down into lower reaches; although the river now normally flows straight through, in time of flood this wooded area still acts as a silt-trap. Of the material that was scoured from the bed of the river below Sardinal a small quantity was deposited in the reaches of the river above Milovon, a larger quantity was deposited in the widening of the channel under Milovon bridge, and most of the remainder was spread out over the swampy land below the bridge. Silt brought down by the river in each flood is deposited in this area below the bridge, but large quantities of silt are still carried into the sea by the flood-waters. Local scouring of the river's banks wherever the soil is friable has continued during each flood-season, and the construction of the usual training works has been necessary each year; but no new tendency to looping has yet occurred, which can be attributed to the radical alterations which have been effected in the regime of the river.

River-training works.—Bank-protection was effected where necessary either by continuous revetment or by the construction of spurs. Continuous revetment formed of wire-netting bags filled with stones was used on several of the concave banks of the existing bends of the Aliakmon, but experience showed that such revetment must be carried down to a good depth and that a heavy toe must be provided; further, in localities where the soil was very friable the water could easily wash away the ground behind the netting and cause settlement. In the Axios under normal conditions spurs about 65 feet long were found to be satisfactory (the width of the river's channel being about seven to ten times the length of the spurs). They were built perpendicular to the bank or pointed slightly downstream; their spacing was about four times their length when the curvature of the line joining their ends was sharp (with a radius less than, say, 1,500 feet), or when the depth of the channel and the velocity of the river close to the bank were above the normal, and was about six times their length when the curvature was flatter. Protective spurs were generally cheaper and more effective than continuous bank-revetment.

In cases where serious erosion had occurred before protective works could be undertaken, longer spurs were constructed, their lengths depending upon the amount of erosion which had occurred. Silt could be expected to deposit between these spurs to within a distance from the line joining their ends of one-third to one-quarter of the distance between them, and was often deposited up to a level

of 1 foot below the top of the spurs. The most successful spurs were of stones in wire-netting boxes; they needed wide bases, well-extended around the end of the spur, and well-protected roots in the river's bank to prevent high floods eroding behind them. Each spur was constructed up to ground-level for half its length, and the outer half was made gradually decreasing in height. Spurs of lighter construction were effectively used in emergencies and, in some cases, for permanent work.

The river-widenings under the new bridges.—The widenings of the existing channels under the new bridges on the Axios and Aliakmon were originally intended to be excavated down to the level of the river's mean summer flow; actually, however, the excavation was not carried down to that level in all cases. Silting occurred after the widenings were opened to flow. In the widening of the Aliakmon channel under Milovon railway bridge about 320,000 cubic yards of silt was deposited during the first flood-season, in that at the Axios road-bridge over 2 feet of silt was deposited during the first year, and similar deposits occurred at the Axios railway-bridge. The top of the silt was approximately at the level of the river during the months of greatest discharge. It was hoped that in the event of a large flood this silt would be scoured out by the flood-waters sufficiently to give the necessary waterway under the bridges, but it has become evident that these deposits become too consolidated by the rapid growth of grass and bushes to permit reliance being placed on the flood-waters having sufficient time in which to scour them away. This experience suggests that similar widenings should in future be designed with a shallower excavation, a length of bridge long enough to give sufficient waterway without any allowance for scour, and with all the works and embankments upstream of the bridge strong enough to resist the higher water-levels which would result from the afflux caused by the possible entire silting-up of the river-widening.

The roughness-coefficient for flow in channels.—Careful measurements were made to determine the roughness-coefficient in the Axios diversion-channel after opening. Very much smaller values of the coefficient were found in reaches where silt had been deposited than in reaches where scouring was in progress. Three weeks after opening, in April, 1934, the average value in the top reach of the diversion was 0·0287. During May erosion was occurring, and the effect of irregularities in the bed became more pronounced as the discharge lessened. On the 1st June the average value at fifteen cross-sections in the top reach was 0·0317. On the 1st June in the lower part of the diversion the average value at eighteen cross-sections was 0·0212. Extreme values were measured of about 0·038

at one cross-section in the top reach and of 0·014 at one cross-section in the bottom reach, and were accounted for by local peculiarities, by small errors in observation, and by the fact that the channel was not in a stable condition.

The roughness-coefficient in existing channels of rivers and streams varied greatly according to local conditions. Values exceeding 0·038 were estimated in some cases, and a low value of 0·0195 was measured in the river Axios (before diversion), in a straight reach where the bed was smooth and the cross-sections uniform.

Measurements made in the main drainage-channel of lakes Amatovo and Ardzan gave a mean value of 0·0251, and this may be taken as representative of the values of the roughness-coefficient which occurred in the new channels when moderately well maintained.

The reclaimed areas.—To determine the amount of settlement and shrinkage of the reclaimed land in lake Yenitsa, careful levels were taken at regular intervals; to determine the rate at which the soft silty material in the bottom of the lake dried out, the "penetrability" of the soft material was measured by the distance which a rod with a flat plate 10 centimetres square at its extremity could be pushed into this soft material with a certain force. The following figures show the mean measurements taken at ten points at each of four localities in lake Yenitsa:—

Date.	Settlement of surface: inches.	Penetrability: inches.
15 June, 1933	0·0	53
1 August, 1933	12·6	30
1 November, 1933	16·3	24½
1 February, 1934	15·6	23½
1 May, 1934	16·1	17¾
1 August, 1934	17·9	9¾
1 November, 1934	19·9	(Ground firm)
1 February, 1935	19·7	" "
1 May, 1935	20·0	" "

The rate of shrinkage was greatest at the commencement of drying out, and at the end of each summer; in winter the rate was slower, and even some swelling occurred. The greatest original depth, 80 inches, of soft material occurred in the centre of the lake, where the settlement of the surface amounted to 34½ inches by the 1st May, 1935.

Observations were made to determine the movement of the ground-water level at numerous points in the Plain. The following figures show the effect on the ground-water level of draining lake

Yenitsa, as well as the general movement of the level throughout the year :—

Date.	Mean depth of ground-water below ground-level.					
	In the central area of lake Yenitsa.		Near the edge of lake Yenitsa.		Elsewhere in the Plain.	
	ft.	ins.	ft.	ins.	ft.	ins.
June, 1933	—		3	9	6	6
September, 1933	4	0	5	3	7	0
December, 1933	1	0	2	9	4	3
March, 1934	1	3	1	9	4	3
June, 1934	4	0	4	3	5	9
September, 1934	6	9	8	0	7	6
December, 1934	3	3	4	0	5	3
March, 1935	3	6	3	3	3	9
June, 1935	5	6	6	0	—	
September, 1935	6	6	8	0	—	

Excellent results have been obtained where crops have been planted in the reclaimed lands, but no exact figures are available of the agricultural yields. As the result of the works which have now been completed 108,000 acres of land has been drained and a further 198,000 acres has been protected from floods. The total cost of all these works, including the diversion of the Axios, the works of river-training and the improvement of communications, together with the maintenance of the works up to the 31st December, 1935, amounted to $15,660,000. This actual cost represented a large saving on the original estimate of $26,570,000 which was attached to the contract of 1925. Examples of unit costs are given in Appendix VIII, pp. 285–6.

During the work the metric system was used for all measurements and quantities; figures have been converted into their approximate equivalents in English units for the purpose of this Paper. Prices and costs are given in United States dollars (expenditures in other currencies being converted into dollars at the average rate for the month in which the expenditure was incurred). Owing to the great variations during recent years in the exchange-rates of the pound sterling, the dollar and the Greek drachma, no attempt has been made to convert the costs into their sterling equivalents.

The Author was on the staff of the Foundation Company on these works from 1929 to 1936, being at one time Project Engineer and later Construction Superintendent, and succeeding Mr. K. J. C. Hill, B.Sc., M. Inst. C.E., in 1934 as the Company's Agent and Chief Engineer in Greece. He desires to take this opportunity to record

his thanks to the Hellenic Government and to the Foundation Company for their permission to present this Paper.

The Paper is accompanied by five sheets of drawings, from which Plate 1 and the Figures in the text have been prepared, by twenty-four photographs, and by the following eight Appendixes.

APPENDIX I.

Meteorological Records.

Rainfall.

Rainfall in Salonika (average of 35 years): inches.

January	1·62	July	0·95
February	1·30	August	0·94
March	1·52	September	1·48
April	1·59	October	2·31
May	2·16	November	2·22
June	1·48	December	1·88

Total per year . . . . 19·45 inches.

The maximum rainfall in 1 year at Salonika was 30·3 inches in 1900.
,, minimum ,, ,, ,, 10·7 ,, 1932.

Rainfall in Salonika Plain: the mean is about 21·2 inches per year. (Rain falls generally during about 80 days in the year.)

Heavy Rainfall has been recorded as follows:—

1·10 inch in 25 minutes at Amatovo (10.7.26).
1·19 inch in 1½ hours at Salonika (14.6.34).
1·66 inch in 4 hours at Yaladjik (1.11.34).
3·22 inches in 8 hours at Katerini (9.12.35).
4·40 inches in 13 hours at Verria (9.12.35) (most of the rain fell in 4 hours).
6·34 inches in 22 hours at Sermenli (6.10.27).
6·62 inches in 24 hours at Edessa (27.3.28) (most of the rain fell in 6½ hours).
10·30 inches in 36 hours at Katerini (9.12.35).
15·60 inches in 4 days at Katerini (Dec. 35).
24·60 inches in 5 days at Edessa (Mar. 28).

Temperature.

Monthly Shade Temperatures in the Salonika Plain (average of 6 years):

	Maximum: ° F.	Minimum: ° F.	Mean: °F.
January	59	24	42
February	64	22	44
March	74	29	52
April	82	35	58
May	89	45	67
June	99	55	77
July	104	59	82
August	100	61	81
September	96	50	73
October	90	41	65
November	74	34	54
December	64	27	46

Extremes recorded: maximum, 115° F. (23.7.34).
minimum, 5° F. (20.12.25).

Frost.—Frost occurs generally on about 35 days in each year, between the end of November and the end of March.

Wind.—A strong northerly wind, known locally as the *Vardar* wind, may spring up suddenly at any time during the year.

APPENDIX II.

Iszkowski Formulas.

The Iszkowski formulas for low-water, mean and maximum flows were first given in 1886,[1] and were based upon data relating to Austrian and German rivers. The formula for maximum floods is of the form $Q = mhCF$,

where Q denotes the maximum discharge in cubic metres per second,
h ,, mean annual rainfall in metres,
F ,, total catchment-area above the point considered in square kilometres,
m ,, a coefficient relating the decrease in the intensity of run-off with the increase in the catchment-area, for example :—

$F = 1$;	50;	100;	1,000;	10,000;	250,000;
$m = 10$;	7·95;	7·4;	4·7;	3·017;	1·0;

and C denotes a coefficient to be chosen according to (i) the amount of vegetation and the permeability of the soil, and (ii) the land formation, having values, for example, of 0·017 to 0·040 for lowlands, 0·070 to 0·155 or more for hilly land, and 0·160 to 0·600 or more for mountains.

When, as is usual, the catchment includes ground of various categories, the formula is used in the form $Q = mh\Sigma(cf)$, where c and f refer to the separate parts of the catchment.

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[1] "Beitrag zur Ermittlung der Niedrigst—Mittel—und Höchstwassermenge mit Hilfe von der Regenhöhen auf Grund Kennzeichnender Merkmale der Flussgebiete." *Zeit. des Österreichischen Ingenieure und Architekten Vereines*, 1886, p. 69.

APPENDIX III.

Particulars and Cost of Excavators.

(The total cost of all the plant employed was $2,498,000.)

Number.	Particulars of machine.	Ground pressure: lbs. per square inch.	Power.	Horse-power.	Approx. weight: tons.	Cost (each): dollars.
1	25·6-inch suction-cutter dredger	—	Diesel-electric.	1,800	850	231,000
1	(Terminal pontoon and pipe-line)	—	—	—	200	19,000
1	21·6-inch suction-cutter dredger	—	Diesel-electric.	730	500	113,000
1	(Terminal pontoon and pipe-line)	—	—	—	300	49,000
1	Multi-bucket excavator (on rails) (detached engine-house). { Buckets, 14 cubic feet; speed, 26 per minute Conveyor, 130 feet by 41 inches }	Less than 14	Diesel-electric.	475	265	167,000
3	6-cubic-yard draglines. Boom 155 feet	22·5	Diesel-electric.	380	455	192,500
2	3½/5-cubic-yard draglines. Boom 137/125 feet	—	Diesel-electric.	300	200	113,000
2	(Pontoons for draglines.)	—	—	—	145	32,700
2	1¾-cubic-yard draglines. Boom 70 feet	—	Steam (coal).	100	70	27,000
2	(Pontoons for draglines)	—	—	—	60	8,700
3	2½-cubic-yard draglines. Boom 75 feet	12·3	Steam (coal).	100	90	41,500
2	2-cubic-yard draglines. Boom 75 feet	10·8	Diesel.	210	90	43,400
3	1¼-cubic-yard draglines. Boom 60 feet	13·2	Diesel.	110	45	25,000
1	¾-cubic-yard dragline. Boom 35 feet	11·2	Petrol.	55	19	13,600
1	¾-cubic-yard dragline. Boom 35 feet	12·5	Diesel.	63	21	11,200
4	¾/1-cubic-yard cranes. Boom 50 feet	16·5	Steam (coal).	140	50	17,000

Note.—All cranes and draglines on land are mounted on caterpillar tracks.

APPENDIX IV.

Outputs and Operation of Excavators.

Machine.	Period.	Percentage of actual digging hours to total hours worked.	Average output per digging hour: cubic yards.	Average output per machine per month: cubic yards.	Maximum output of one machine in one month: cubic yards.	Average fuel-consumption per month per machine.	Average lubricating-oil consumption per month per machine: lbs.
25·6-inch suction-cutter dredger (diesel-electric).	1933–1934, 11 months.	73	850	424,000	610,000	103 tons diesel oil.	12,250
21·6-inch suction-cutter dredger (diesel-electric).	1931–1934, 34 months.	75	296	113,000	221,000	39·4 tons diesel oil.	2,950
Multi-bucket excavator, on rails (diesel-electric).	1931–1934, 40 months.	68	320	115,000	220,000	22·7 tons diesel oil.	3,750
6-cubic-yard draglines, on caterpillar tracks (diesel-electric).	1930–1934, 140 machine-months.	61½	184	65,100	173,000	20·75 tons diesel oil.	2,450
3½/5-cubic-yard draglines, on pontoons (diesel-electric).	1930–1935, 116 machine-months.	64½	164	55,400	112,500	15·5 tons diesel oil.	1,805
1¾-cubic-yard draglines, on pontoons (coal-burning, steam).	1931–1933, 55 machine-months.	57½	77½	22,700	39,000	83 tons coal.	470
2½-cubic-yard draglines, on caterpillar tracks (coal-burning, steam).	1929–1934, 165 machine-months.	65	100	34,300	61,000	96 tons coal.	855
2-cubic-yard draglines, on caterpillar tracks (diesel).	1931–1934, 53 machine-months.	62	90½	33,500	71,500	5·35 tons diesel oil.	585
1¼-cubic-yard draglines, on caterpillar tracks (diesel).	1930–1935, 109 machine-months.	65	57	19,900	40,000	3·2 tons diesel oil.	575
¾-cubic-yard dragline, on caterpillar tracks (diesel).	1935, 6 months.	69	54	22,300	29,500	2·3 tons diesel oil.	305
¾-cubic-yard dragline, on caterpillar tracks (petrol).	1932, 6 months.	70	29	12,200	16,400	1,000 galls. petrol.	285
¾-cubic-yard cranes, 50-foot jib (coal-burning, steam).	1931, 14 machine-months.	65	29½	11,100	20,900	37 tons coal.	400

APPENDIX V.

Running Costs of Excavators.

Machine.	Mounting.	Power.	Period.	Total cost: dollars per month.	Unit costs: cents per cubic yard.			
					Fuel and lubricants.	Repairs and renewals.	Labour on machine.	Total.
25·6-inch dredger	—	Diesel-electric.	1933–34	5,500	0·57	0·41	0·31	1·29
21·6-inch dredger	—	Diesel-electric.	1931–34	3,000	0·73	0·69	1·24	2·66
Multi-bucket excavator	On rails.	Diesel-electric.	1931–34	2,700	0·57	0·91	0·84	2·32
6-cubic-yard dragline	Caterpillars.	Diesel-electric.	1930–34	1,950	0·85	1·19	0·93	2·97
3½/5-cubic-yard dragline	Pontoon.	Diesel-electric.	1930–35	1,800	0·78	0·96	1·51	3·25
1¾-cubic-yard dragline	Pontoon.	Steam (coal).	1931–33	1,150	2·28	1·09	1·63	5·00
2½-cubic-yard dragline	Caterpillars.	Steam (coal).	1929–34	1,700	1·99	1·47	1·42	4·88
2-cubic-yard dragline	Caterpillars.	Diesel.	1931–34	770	0·45	1·01	0·83	2·29
1¼-cubic-yard dragline	Caterpillars.	Diesel.	1931–35	590	0·57	1·22	1·15	2·94
¾-cubic-yard dragline	Caterpillars.	Diesel.	1935	380	0·36	0·62	0·73	1·71
¾-cubic-yard dragline	Caterpillars.	Petrol.	1933	390	1·45	0·85	0·85	3·15
¾-cubic-yard crane	Caterpillars.	Steam (coal).	1934	600	2·12	0·99	2·24	5·35

APPENDIX VI.—Particulars and Cost of Bridges.

(The aggregate length of the bridges constructed is 12,700 feet. Bridges under 50 feet long are not tabulated.)

Name.	Spans.	Total length: feet.	Superstructure.	Foundations.	Approximate cost. Total: dollars.	Approximate cost. Unit: dollars per foot.
Railway Bridges:						
Axios	17	2,086	Warren-truss steel spans on mass-concrete piers.	30-foot timber piles. Max. load 14·2 tons.	342,400	164
Vardarovassi	5	550	(Skew) warren-truss steel spans on mass-concrete piers.	30-foot timber piles. Max. load 13·6 tons.	90,200	164
Milovon (over Aliakmon)	6	995	(Elongation of existing bridge) Baltimore-truss steel spans on mass-concrete piers.	72-foot timber piles. Max. load 14·6 tons.	289,400	291
Milovon (temporary)	7	387	Plate-girders on timber piers.	Timber piles.	40,000	103
Kouloura (over Circulatory canal).	9	625	(Skew) plate-girders on mass-concrete piers.	Concrete slab (with steel sheet-piling), 1·9 tons per square foot.	149,500	239
Loudias	1	140	Warren-truss steel span on r.-c. abutments.	39-foot timber piles. Max. load 16·3 tons.	37,000	265
Limnotopos sluice	3	65	Steel girders on concrete piers.	Mass-concrete slab and timber piles.	114,800	—
Main Road-Bridges:						
Axios	14	1,853	Warren-truss steel spans on mass-concrete piers.	39-foot timber piles. Max. load 15·5 tons.	216,400	117
Verria (over Circulatory canal).	3	404	Warren-truss steel spans on concrete piers.	Timber and r.-c. piles. Loads 18 and 39 tons.	66,500	165
Nishel (over Aliakmon)	5	348	Old steel girders on concrete piers.	38-foot timber piles. Max. load 10 tons.	34,800	100
Loudias	1	160	Warren-truss steel spans on r.-c. abutments.	39-foot timber piles. Max. load 18·1 tons.	33,500	210

Loudias pontoon (temporary)	—	340	Timber trestle; removable span on timber pontoons.	25-foot timber piles. Max. load 10 tons.	21,100	62
Vardarovassi	15	364	Reinforced concrete.	30-foot r.-c. piles. Max. load 19 tons.	31,900	88
Balitsa	8	200	Reinforced concrete.	33-foot r.-c. piles. Max. load 30 tons.	19,800	99
Limnotopos	3	121	Reinforced concrete.	30-foot r.-c. piles. Max. load 25 tons.	32,300	267
Vardaroftsa	3	121	Reinforced concrete.	30-foot r.-c. piles. Max. load 25 tons.	36,300	300
Upper Notched weir (over Circulatory canal).	8	328	Reinforced-concrete bridge over weir.	R.-c. slab. Load 1·8 ton per square foot.	38,000	—
Lower Notched weir (over Circulatory canal).	5	165	Reinforced-concrete bridge over weir.	33-foot timber piles. Load 12–15 tons.	113,100	—
Smaller Road-Bridges:						
Selimli	4	127	Reinforced concrete.	R.-c. piles.	4,400	35
Koulakia (over Axios)	14	368	Timber deck on reinforced-concrete columns.	39-foot r.-c. piles. Max. load 17 tons.	11,400	31
Valmada (over Axios)	19	468	Reinforced-concrete.	R.-c. piles.	14,000	30
Diavatos (over Circulatory canal).	16	390	Timber deck on reinforced concrete columns.	R.-c. piles.	12,800	33
4 timber bridges (over Circulatory canal), each.	14	356	Timber.	Timber piles. Load 9 tons.	8,300	23
Kourfalia (Vardarovassi channel).	11	275	Timber.	Timber piles. Load 9 tons.	5,300	19
Ardzan	5	75	Timber.	Small timber piles. Load 4 tons.	2,400	32
Foot-Bridge:						
Eleoussa (Vardarovassi channel)	25	282	Reinforced-concrete deck, on old rails as columns.	Old rails as piles.	1,200	4·3

Notes.—Railway bridges were single-track, and were designed for two 135-ton locomotives with 15-ton waggons or for two axle-loads each of 22 tons. Main road-bridges were about 20 feet wide, and were designed for a 14-ton roller or truck, or 102 lbs. per square foot. Tertiary road-bridges were about 13 feet wide, and were designed for a 7-ton truck or 82 lbs. per square foot. The costs given do not include general head-office expenditure, engineering, or profit.

APPENDIX VII.—Some Estimated Discharges of Actual Floods.

River.	Catchment-area: square miles.	Original design discharge: cusecs.	Estimated actual discharge: cusecs.	Date of flood.	Estimated actual run-off: cusecs per square mile.	Rainfall at the nearest gauging-station: inches.
Axios	8,580	134,000	123,000	1907	14·3	—
Axios	8,580	134,000	86,200	11.12.35	10·05	—
Aliakmon (at Milovon)	3,400	121,700	142,500	10.12.35	42	—
Aliakmon (at Kokovo)	2,540	76,200	116,500	10.12.35	46	4·27 in 1 day (average of 5 stations).
Aliakmon (at Kokovo)	2,540	76,200	65,300	28. 3.28	25·7	—
Circulatory canal (at outlet)	780	55,500	41,500	10.12.35	(53)	—
Circulatory canal (at inlet)	545	42,300	29,600	10.12.35	(54)	—
Moglenitsa	385	38,300	30,000	10.12.35	78	5·93 in 1 day at Edessa.
Galikos	385	24,700	22,400	10.12.35	58	2·18 in 6½ hours at Salonika.
Voda	101	11,330	12,950	27. 3.28	128	6·61 in 1 day at Edessa (most fell in 6½ hours).
Voda	101	11,330	10,020	9.12.35	99	5·93 in 1 day at Edessa.
Ayak	100	4,940	6,100	10.12.35	61	2·64 in 1 day at Amatovo (most fell in 8 hours).
Arapitsa	65·6	12,320	9,240	10.12.35	141	5·50 in 1 day (average of Edessa and Verria).
Ana Dere	64·0	12,350	22,400	10.12.35	350	Over 4·30 in 13 hrs. at Verria (most fell in 4 hours).
Kutika	48·6	8,330	11,300	10.12.35	232	Over 4·30 in 13 hrs. at Verria.
West Balitsa	44·5	3,880	4,130	10.12.35	93	—
Selimli	35·9	2,820	3,950	10.12.35	110	2·64 in 1 day at Amatovo (most fell in 8 hours).
West Moglenitsa	25·1	—	4,580	9.12.35	183	5·93 in 1 day at Edessa.
Tchinarli	21·6	3,000	2,080	10.12.35	96	1·48 in 6 hours at Yaladjik.
Rizovo	16·2	—	2,650	9.12.35	164	5·50 in 1 day (average of Edessa and Verria).
Tchekere	15·4	2,290	1,800	10.12.35	117	1·48 in 6 hours at Yaladjik.

APPENDIX VIII.

EXAMPLES OF UNIT COSTS.

Excavation in Cofferdam (including use and waste of temporary materials).—	Dollars per cubic yard. Labour.	Materials.	Total.
Axios railway bridge	2·25	2·62	4·87
Axios road-bridge	1·11	1·41	2·52
Kouloura railway bridge	0·81	0·91	1·72
Verria road-bridge	0·89	1·30	2·19
Concrete in Foundations.—			
Axios railway bridge	1·10	5·20	6·30
Axios road-bridge	0·48	3·29	3·77
Verria road-bridge	0·34	2·89	3·23
Upper Notched weir	0·31	2·78	3·09
Concrete in Superstructures (including cost of shuttering).—			
Axios railway bridge	1·06	6·09	7·15
Axios road-bridge	1·07	3·45	4·52
Verria road-bridge	1·52	3·59	5·11
Upper Notched weir	0·86	4·17	5·03
Reinforced Concrete (including cost of shuttering).—			
Lower Notched weir: deck	6·05	8·56	14·61
Balitsa bridge: abutments	3·20	5·75	8·95
decking	9·02	8·16	17·18
Upper Notched weir: deck	1·85	8·25	10·10
Diavatos road-bridge	5·23	9·45	14·68
Timber Piles in Foundations.—	Dollars per linear foot. Driving.	Timber.	Total.
Axios railway bridge	0·72	0·73	0·85
Axios road-bridge	0·11	0·51	0·62
Verria road-bridge	0·09	0·58	0·67
Loudias railway bridge	0·07	0·55	0·62
Timber Sheet-Piling.—	Dollars per square foot. Labour.	Materials.	Totals.
Ayak and Selimli weirs	0·126	0·132	0·258
Vardarovassi weirs	0·028	0·136	0·164
Tchinarli and Tchekere	0·035	0·126	0·161
Circulatory canal siphons	0·054	0·203	0·257

	Labour.	Materials.	Total.
Reinforced-Concrete Piles: Driving Only.—	Dollars per linear foot.		
Vardarovassi road-bridge	0·225	0·107	0·332
Balitsa road-bridge	0·232	0·101	0·333
Koulakia road-bridge	0·067	0·089	0·156
Diavatos road-bridge	0·130	0·360	0·490
Masonry.—	Dollars per cubic yard.		
Vardarovassi canal falls	0·64	1·42	2·06
Circulatory canal infalls	0·99	2·56	3·55
Circulatory canal siphons	1·59	2·65	4·24
Buildings	0·72	1·92	2·64
Pitching Pointed with Mortar.—			
Vardarovassi canal infalls	0·52	1·35	1·77
Tchekere falls	0·59	1·43	2·02
Circulatory canal infalls	0·92	1·98	2·90
Circulatory canal siphons	1·02	2·16	3·18
Dry Stone Pitching.—			
Vardarovassi canal falls	0·58	0·61	1·19
Tchekere falls	0·60	1·12	1·72
Sardinal dam	0·40	1·08	1·48
Circulatory canal siphons	0·98	1·78	2·76
Stones in Wire-Netting Bags.—			
River Axios spurs	0·79	3·05	3·84
Selimli weir	0·50	2·34	2·84
Aliakmon bank pitching	0·36	3·66	4·02

Road-Construction.—	Dollars per square yard.
Service roads	0·33
Main roads: Axios road-bridge, approaches	0·69
Nishel–Milovon road	0·86
Upper Notched weir, approaches	0·65

Bush-Clearing.—	Dollars per acre.
Aliakmon floodway	38·6

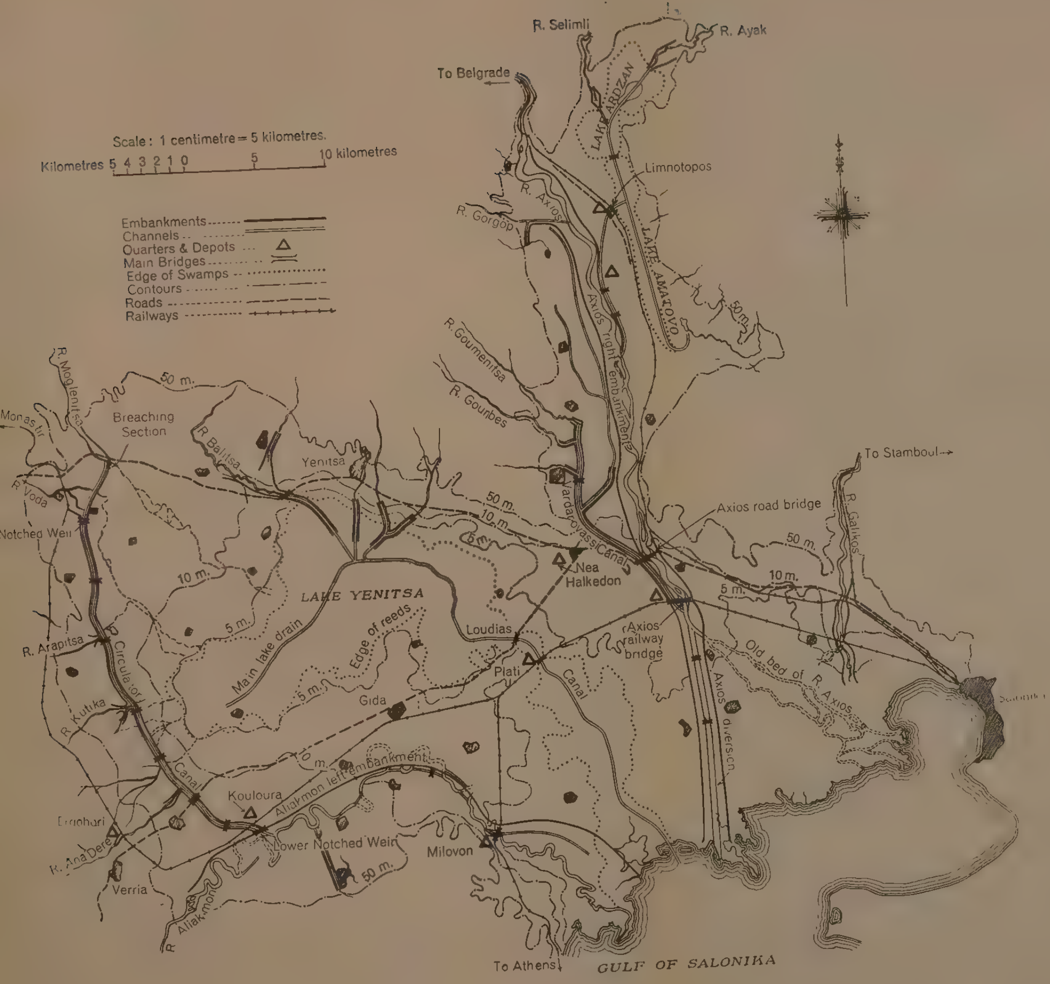
Embankment-Construction.

(Average of 85 miles of protective embankment; over 60 per cent. of this work was done by machines; costs do not include plant-depreciation.)

	Cubic yards: per mile.	Dollars: per mile.
Stripping top soil (including replacing on embankment and sowing grass, etc.)	13,200	1,500
Excavation and formation of embankment	91,500	7,100
Supervision and service works		1,100
Total cost per mile		9,700

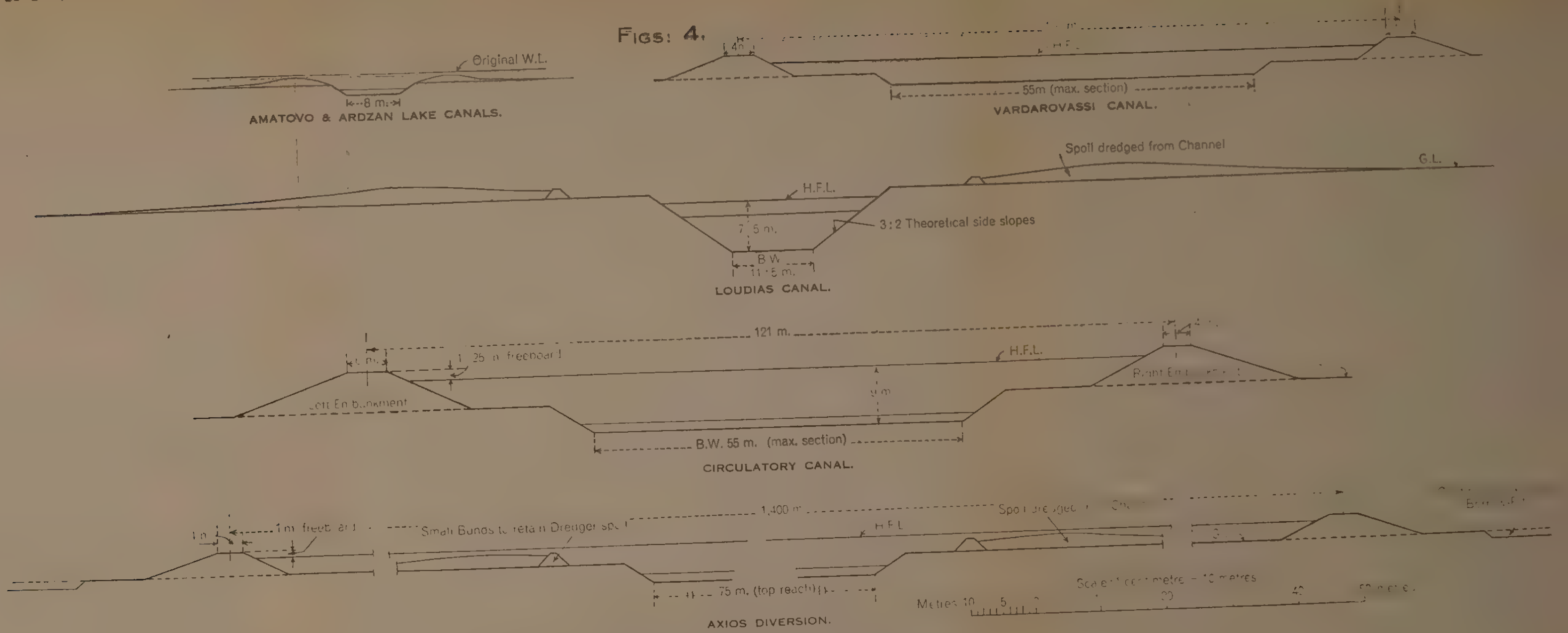
SALONIKA PLAIN RECLAMATION-WORKS.

Fig: 1.



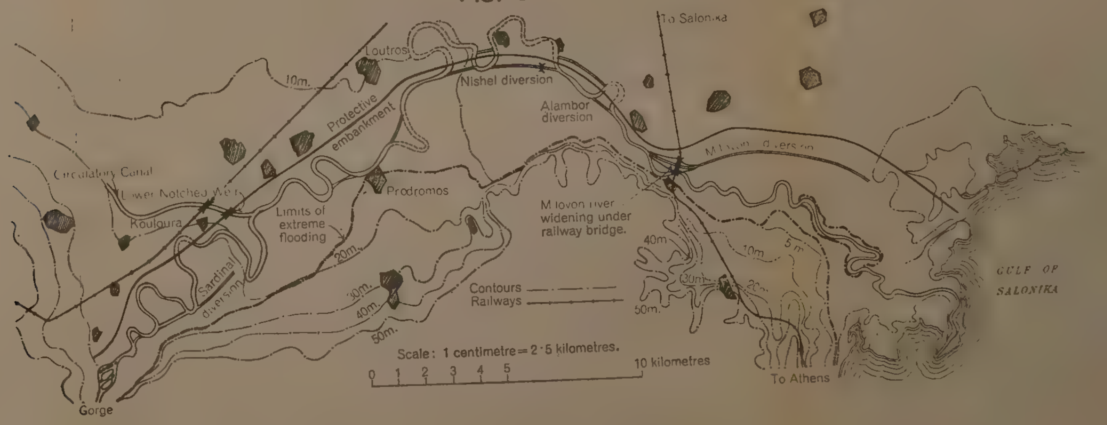
GENERAL MAP OF SALONIKA PLAIN.

Figs: 4.



CROSS SECTIONS OF CHANNELS.

Fig: 6.



RIVER ALIAKMON.

B. W. HUNTSMAN.

William Clowes & Sons, Limited: London.

Paper No. 5097.

"The Lake Copais, Boeotia, Greece: Its Drainage and Development."[1]

By Alec James Dean, B.Sc. (Eng.), Assoc. M. Inst. C.E.

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Introduction.

The Copaic Plain lies in the province of Boeotia, Greece, about 50 miles to the north-west of Athens (*Fig. 1*, p. 288). It is a natural basin about 130 square miles in area, entirely surrounded by hills and mountains. The Plain itself is about 100 metres above sea level. No natural outlet exists above ground for the waters that flow into the Copaic Plain, and, apart from a period in prehistoric times when the Plain appears to have been drained, this basin was largely a lake of varying depth and area until recent years when the works, which it is the main object of this Paper to describe, were executed.

The geological formation of the whole district is principally limestone, and water from the lake escaped down fissures in the limestone round its edges, preventing the water ever rising high enough to form an open outlet. These swallets are a feature of the district and are known as *katavothras*. A line can be drawn across the Copais dividing springs (to the west) from the *katavothras* (to the east). During the summer the discharge of the *katavothras* exceeded that of the rivers flowing into the Copais and the water-level fell; in winter the reverse was the case, and the marsh became a lake again. The condition of the plain and the water-levels also varied considerably over years, probably due to the periodic blocking and clearing of the *katavothras*.

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[1] Correspondence on this Paper can be accepted until 15th July, 1937.—Sec. Inst. C.E.

The present climate of the Copais is somewhat extreme. Temperatures of 100° F. in the shade are common during the summer and frosts occur in the winter. The rainfall is moderate (about 30 inches per annum), but irregular; there is very little rain in

Fig. 1.

THE COPAIC PLAIN, BOEOTIA.

summer, but 7 inches have been recorded in one week in the winter. The question of floods is consequently one of the major problems in the drainage of the Plain.

The object of this Paper is to describe briefly what are supposed

to have been the ancient attempts to drain the Copaic plain, to explain the modern drainage system in more detail, and principally to describe the improvement works which were later found necessary and with the execution of which the Author was connected.

The Ancient Drainage.

References in classical times to pre-Dorian Boeotia mention the fertility of the Copais. The remains of an ancient drainage system found since the modern system of drainage was constructed indicate an extensive and probably successful system of the prehistoric Minoan period (2,000 B.C.). This drainage-scheme appears to have been to intercept the waters from the rivers which fell into the lake and to conduct them by embankments to the *katavothras* in the north-east and south-east areas of the Plain (Fig. 2, Plate 1). Two main canals were built, the northern canal intercepting the Kefissos and the Melas rivers and the southern canal intercepting the waters from the rivers off Mount Helicon in the south west. An interesting point about the Minoan embankments is that they were constructed with either a revetment or a core of cyclopean masonry, and were of remarkable size. Part of the northern bank is to-day 68 metres wide and is reinforced by two polygonal walls, 2 metres thick and 27 metres apart. A discharge-canal, leading to the Binia and Grand *katavothras*, the two largest *katavothras* in the north-east bay, is also remarkable in size. Signs are found of other attempts made after these works fell into disuse, probably due to war, but they do not appear to have been successful.

Drainage under the French Company.

Macedonians, Franks, Catalans, Venetians, and Turks have all left their mark in the district, but no records exist of attempts to drain the Plain. Shortly after the War of Independence, various projects were formed, beginning in 1846 when plans for the drainage of the Plain were drawn up by Monsieur Sauvage. A French company was formed in 1880 for the reclamation of the lake, revised plans being drawn up by Monsieur Tarratte in 1881, after whose death Monsieur Léon Pochet completed them and carried them out in the years 1882 to 1886. This company's project consisted essentially of the following works :—

(1) A "girdle canal," now called the Grand canal (33 kilometres in length), to carry the water of the Kefissos river round the south side of the lake and to intercept the

various rivers that flow into it from the surrounding hills.

(2) The canalization of the Melas river, leading it around the north side of the lake to meet the Grand canal at Karditsa.

(3) The Interior canal to take the run-off from the actual rainfall on the Plain itself and to join the other two canals at Karditsa.

The water from these three systems was to be conducted to the sea via lakes Likeri and Paralimni by the following further works :—

(4) A deep cutting, 2,760 metres long, called the Emissary canal, revetted with stone, from Karditsa (at the outfall of the three systems mentioned above) towards the hills.

(5) The Karditsa tunnel (672·50 metres long, 9·50 metres high, and 6·50 metres wide), cut through a spur of Mount Ptoon, to lead the waters of the Emissary canal to lake Likeri.

(6) A conduit at Moriki to carry the overflow from lake Likeri to lake Paralimni.

(7) The Anthedon tunnel (860 metres long) to carry the water from lake Paralimni to the sea.

The arrangement of these works is shown in Figs. 2 and 3, Plate 1. The reason for adopting this expensive route in preference to the much cheaper one by Larymna, where only one tunnel would have been necessary, was that it was intended to use the two lakes as reservoirs and the power of the waterfalls to pump water back to the Copais for irrigation. The French scheme also included a tunnel (1,030 metres long) at Hungara.

The Karditsa tunnel was opened officially in 1886, and by that time the Emissary canal at Karditsa and the Anthedon tunnel had been completed, the Moriki cut was nearly completed, the Grand canal had been started and 300 metres of the Hungara tunnel had been completed, although this tunnel was never finished.

The Work of the British Company.

The French company, however, failed, and in the following year, 1887, the works were taken over by the present British company. The project of the French company, except the Hungara tunnel and its irrigation-scheme (which were not considered practicable), was completed by the British company. These works are shown on Fig. 2, Plate 1.

In addition to works already mentioned, the Grand canal (and its extension, the Marsh canal) had been completed, including the Grand

barrage, built across the bed of the old Kefissos in order to divert the water down the Grand canal; the Melas canal bank had been built; and the Melas canal and the Interior canal had been excavated.

In explaining the evolution of the drainage-works from the French project to the position to-day it will be clearer to do so under the two headings of exterior drainage, and interior drainage and development.

Exterior Drainage.

The original French project was quite sound as regards the exterior drainage of the Copais, although the capacities given to the canals have proved to be rather on the small side. It will be seen from Table I, which shows the estimated flood-discharge of the various

Table I.—Capacities, etc. of Exterior Drainage Works According to Monsieur Pochet's Scheme of 1882.

Point in canal.	Flood discharge of river entering canal: cubic metres per second.		Discharge of overflow weir or escape sluice when canal is flowing to full capacity: cubic metres per second.		Capacity of canal: cubic metres per second.
Grand canal:					
Kil. 23 . .	Kefissos	100	Grand barrage	10	90
Kil. 20 . .	—		Weir	5	—
Kil. 19 . .	Hercyna	100	Weir	50	110
Kil. 17 . .	—		Weir	5	—
Kil. 12 . .	Ponjia	80	Weir	10	125
Kil. 7 . .	Vrastamites	5	—		—
Kil. 5·7 . .	Siacho	15	—		—
Kil. 0 . .	Lophis	40	—		—
Marsh canal . .	—		—		130
Emissary canal .	—		—		130
Tunnel . . .	—		—		140

The above capacities were calculated with the water flowing 50 centimetres below the tops of the banks of the Grand and Marsh canals.

rivers flowing into the Grand canal, the designed capacity of the various reaches of this system, and the capacity of the various overflow-weirs, that the capacities of the various sections of the Grand canal, allowing for the overflows provided in the left bank, are quite inadequate to carry the flood-discharges of the rivers flowing into it, if they were all to arrive at once. Monsieur Pochet had, however, correctly judged that the floods of the local rivers would always have passed before the Kefissos flood arrived.

During the early years considerable trouble was experienced with the banks of the Grand and Marsh canals, and breaks occurred on

several occasions with consequent extensive flooding. Following the advice both of Monsieur Pochet and the late Mr. C. S. Meik, M. Inst. C.E., who was also consulted at that time, several works were executed in the years that followed, further to strengthen and safeguard the exterior drainage-works. These works consisted principally of raising the banks of the Grand canal, lengthening an overflow weir in the left bank of the canal, improving the right guard drain, providing additional automatic doors, and making the sectional area of the canal to approximate to the original sections planned by Monsieur Pochet. Both banks of the Marsh canal were also raised to a level 40 centimetres above high-flood level, and sluice-gates were built in the right bank to act as an escape in time of flood and also for the purpose of warping the land to the south of this canal.

Mr. Meik also recommended that high floods from the Kefissos should be relieved by opening the Grand barrage on the old Kefissos. He suggested that the water thus escaped should be carried into the Melas river and then either to the Grand *katavothra* or to the Emissary canal via the Melas canal, or to both. This scheme was later modified and a siphon was constructed under the Melas to carry this water into the basin of Division " L " (Fig. 3, Plate 1). The Great War delayed the execution of this scheme, which was eventually completed in 1921.

In his report Mr. Meik also gave his opinion that the capacity of the Emissary canal was insufficient, and that had the Emissary canal and tunnel been originally kept at a lower level by from 1 to 2 metres the interior drainage would have been very much improved. This is referred to on p. 296.

Fig. 3, Plate 1 shows the position after the execution of the above works, and the principal characteristics of the main exterior drainage works are shown on Figs. 4, Plate 1. Except at its downstream end, where it passes through high land before entering the Emissary canal, the Grand canal consists of a cut, below the level of the surrounding land, of a capacity sufficient for normal discharges, the spoil from this cut being used to form banks which are thrown back a considerable distance so as to give the channel from bank to bank a capacity sufficient for flood-discharges. The width from bank to bank varies from 40 to 70 metres and the width of the bed of the cut from 9 to 22 metres, the gradient varying from 17 to 50 centimetres per kilometre. This canal has proved generally stable. Little scour or silting has occurred in the lower channel, although some silting is noticeable on the berms in various lengths. The capacities of the various stretches of this canal, as intended by Monsieur Pochet in 1882, are shown in Table I (p. 291); Table II shows them as they were in 1919.

TABLE II.—CAPACITIES OF EXTERIOR DRAINAGE WORKS IN 1919.

Point in canal.	Flood-discharge of river entering canal: cubic metres per second.		Discharge of overflow weir or escape sluice with canal flowing to full capacity: cubic metres per second.		Capacity of canal: cubic metres per second.
Grand canal:					
Kil. 23 . .	Kefissos	150	Grand barrage	6	144
Kil. 20 . .	—		Weir	5	—
Kil. 19 . .	Hercyna	100	Weir	55	150
Kil. 17 . .	—		Weir	5	—
Kil. 22 . .	Ponjia	90	Weir	18	176
Kil. 7 . .	Vrastamites	5	—		—
Kil. 5·7 . .	Siacho	15	—		—
Kil. 0 . .	Lophis	60	—		—
Marsh canal . .	—		—		172
Kil. 8·5 . .	—		Sluice	20	—
Emissary canal .	—		—		131
Tunnel . . .	—		—		149

The above capacities were calculated with the water flowing 40 centimetres below the tops of the banks of the Grand and Marsh canals. The capacity of the tunnel was unobtainable.

The problems that still faced the Company were that the capacities of the Emissary canal and the tunnel were not sufficient for very large floods, and that the interior drainage, although improved by pumping (as explained later), was unsatisfactory. It was felt that any further works should be comprehensive and, as far as possible, final. Various schemes were evolved, and eventually a scheme was approved for deepening the tunnel and widening and deepening the Emissary canal so that they should be both large enough for the highest flood and deep enough to permit gravitational drainage of the interior with very little interruption. Besides these outlet works the scheme provided for a new Great Central drain to drain the lowest land, and for remodelling the Melas canal and river and the Interior canal. These works are described on pp. 294–300.

INTERIOR DRAINAGE AND DEVELOPMENT.

Although the exterior drainage-works of the French project proved generally sound, the French scheme for the drainage of the interior soon proved inadequate. The prime trouble with the interior drainage arose from the fact that a very large area of the old lake-bed sank considerably. This was due principally to the burning of the peat that had formed over all the semi-permanent marsh areas, the likelihood of which had not apparently been considered in the French scheme. This was a factor of the first importance, for the

peat layer was in places nearly 4 metres deep. As the peat burned and the land sank in consequence, the levels changed, and it became evident that the lowest land would be (as now, with almost all the peat burned, it is) in the centre of the old lake and at least 2 metres lower than it had been before. As a result of this fall in the level of much of the plain, large areas remained undrainable by means of the tunnel, and recourse was had to pumping, but, in spite of increasing the capacity of the pumping stations, the position continued to be unsatisfactory.

In the meantime the estate had been developed and barrages had been built on the Grand and Marsh canals to impound the water of the Kefissos, and also in the river Melas, so that the water available in summer could be used for irrigation.

A general canalization of the Plain was also carried out for drainage purposes. The main drains ran from north to south and were collected by a Deep Level drain leading, by a siphon under the Interior canal, to the new pumping stations (Fig. 3, Plate 1). By an arrangement of sluices in the pump canals it was possible to pump with either or both pumping stations from either or both the Interior and Deep Level drains.

Drainage-Improvement Works.

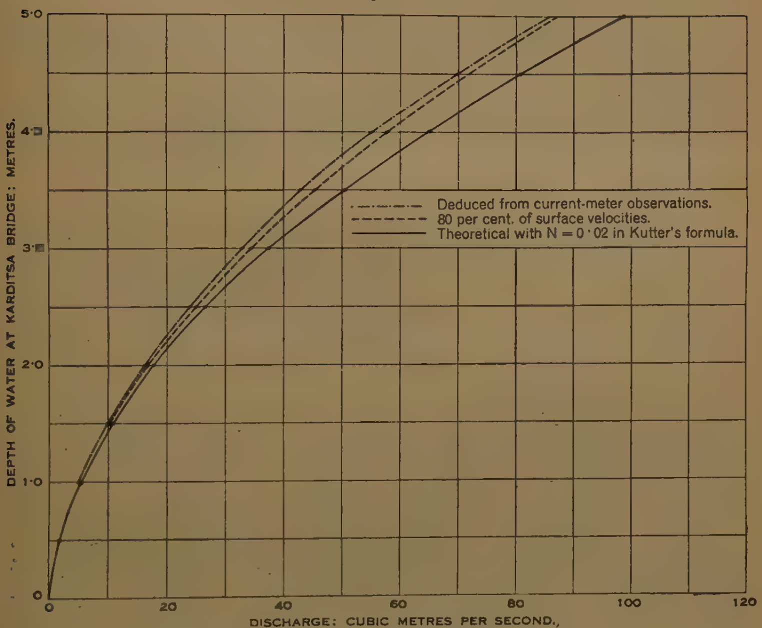
After considerable study, both during and just after the Great War, works were approved to lower and enlarge the outlets from the Copais, the Emissary canal and tunnel, so that they should be large enough to carry all exterior floods and low enough to allow gravitational drainage of the interior. The Interior canal was to be remodelled and a Great Central drain was to be dug from the lowest land right across the centre of the Copais from west to east.

Design.

It was considered that, after the works executed between 1904–1919, the capacity of the Grand canal was sufficient, and that the minor works then in hand on the Marsh canal would make its capacity also sufficient. The discharge of the Emissary canal had been kept regularly since 1896, based on a series of observations made over a long period by Mr. R. S. Cole, Assoc. M. Inst. C.E., when Chief Engineer to the Company, and it was reckoned that its maximum capacity was only about 130 cubic metres per second, whereas the Marsh canal was capable of carrying about 180 cubic metres per second; and experience of large floods went to show that such a quantity was the maximum flood discharge to be expected. It was

therefore decided to increase the capacity of the Emissary canal and tunnel to approximately 180 cubic metres per second. It may be noted here that after the drainage-improvement works had been completed, the Author had a series of discharge-measurements taken in the Emissary canal, which tended to confirm the figures of Mr. Cole; from them it would appear that a coefficient of $n = 0{\cdot}02$ in Kutter's formula was accurate for a stone-pitched channel of the

Fig. 5.



EMISSARY CANAL DISCHARGES.

type in use. The results of these observations are shown by the curves in *Fig. 5.*

In connexion with the exterior drainage-works, the following figures are of interest:—

Total drainage-area above Karditsa tunnel . . .	800 square miles.
Mean annual rainfall in 28 years	28·33 inches.
Maximum ,, ,, ,, ...	41·16 ,,
Mean annual evaporation (water surface) in 4 years .	52·29 ,,

After making provision for the exterior drainage-system to be of sufficient capacity, the general basis of the design of the works may be summarized by stating that the mean winter discharge of the Emissary canal is 35·5 cubic metres per second and that the water-

level at the head of this canal, when taking this discharge, should be such as to allow free flow from the interior canals. This discharge is made up of 28 cubic metres from the Marsh canal, 3 cubic metres from the Melas, 2·25 cubic metres from the Interior canal and 2·25 cubic metres from the new Great Central drain. The first two figures were taken from actual records. The mean winter discharge of the interior drains was determined as follows :—

Great Central Drain—

Maximum daily rainfall . .	0·0084 metre (based on the wettest 2 months on record—October and November, 1918).
Run-off	35 per cent.
Area drained	136,800 square metres.
Maximum discharge . . .	4·26 cubic metres per second.
Mean winter discharge . .	2·13 cubic metres per second.

Interior Canal—

Area drained	75,000 square metres.
Maximum discharge . . .	2·52 cubic metres per second from this area + 1·0 cubic metre per second from Rachi siphon = 3·52 cubic metres per second.
Mean winter discharge . .	1·26 cubic metre per second from this area + 1·0 cubic metre per second from Rachi siphon = 2·26 cubic metres per second.

The most important point was to decide the water-level at the head of the Emissary canal under these conditions. The possible future shrinkage of the land was an important factor, but on the other hand the depth of the cut of the Emissary canal at its deepest part was already very great (18 metres) and the question of cost and time of execution had to be considered. As finally executed this level was fixed at 89·24 metres and the corresponding bed-level at 86·64 metres, the lowest land in the future being expected to be at a level of about 91·00 metres and to lie 10 kilometres distant (Fig. 3, Plate 1).

In view of the great depth of the cut in the Emissary canal it was decided to provide a berm 3·0 metres wide on each side above flood-water level and to lay back the sides above this level to a slope of 1½ to 1, the stone-pitched channel actually carrying the water having its sides at a slope of 1 to 1. Regulators were provided at the outfall of the Melas canal, Interior canal and Great Central drain to prevent exterior flood water heading back up these channels. Perhaps the most delicate point of the whole project was to determine what would happen when the regulator on the Great Central drain was shut at such times. Records of past floods indicated that at a maximum the Great Central drain regulator might be closed for as long as 13 days; experience has, however, shown that on several

occasions in late winter (February or March), with the land already soaked, flooding of the lowest land begins to occur within 48 hours of shutting the regulator. The capacities of the various channels were calculated with Bazin's formula $v = c\sqrt{Ri}$, in which c, with metric units, was taken as 45 for the canals in earth and as 62 for the Emissary canal and tunnel.

As the general bed of the Copais sank, the Melas river, for a great part of its length, was left at a higher level than much of the land. This resulted in considerable infiltration from the river taking place through the porous layer of burnt peat, which made the drainage of large areas extremely difficult. The object of deepening the Melas canal and river was to lower its level below the porous layer as far as possible and so to minimize the infiltration. The good results achieved are described later.

Execution.

In determining the method of executing the works there were three important factors to be considered: first, that work in the tunnel and in the bed of the Emissary canal could only be done for 2 (or at a maximum 3) months of the year, which would be, as a rule, the end of June, July, and August; second, that as much benefit as possible should be obtained as quickly as possible; third, that the remodelling of the Interior canal and of the Melas river and canal, and the excavation of the Great Central drain, would entail work in water.

It was decided to push forward, with as little interruption as possible, with the widening of the Emissary canal, which would give relief in flood-time. This work was carried out with 1-cubic-yard tip-wagons running on a 60-centimetre-gauge line, the spoil being run up ramps in the side of the canal to spoil-heaps on its edges. Three steam locomotives were used for haulage. This plant was also available for work in the tunnel and in the bed of the canal during the short dry season, and assured its expeditious execution. Later, when the bulk of the work was done and the final work became more congested and the economic train smaller, two small petroleum locomotives were also acquired.

For the interior earthworks two Ruston dragline excavators were employed, one a No. 60, with a 60-foot boom and a 1½-cubic-yard bucket, and the other a No. 10, with a 40-foot boom and 1¼-cubic-yard bucket. These machines did their work very well, the cost in each case working out at about 6*d*. per cubic metre. The smaller machine has since travelled hundreds of miles in the Estate on various maintenance works and has proved its usefulness in scores

of ways. The work was executed departmentally, a system of piece-work contracts being employed, under the control of the Company's engineering staff.

The material required for the works was largely available locally. Stone was quarried at Karditsa and elsewhere for the various structures, stone pitching, etc., the whole basin being surrounded by limestone cliffs where excellent quarries can be opened. Sand and gravel were obtainable from the local rivers, and supplies were also taken from gravel pits at Karditsa. Timber in Greece is almost entirely imported, principally from Roumania and Russia, and is quite cheap. Local cement, manufactured either at Piræus or Chalkis, was used throughout the works. It is quite a good cement, comparable with Portland cement. In the early days, pozzuolana was largely employed for hydraulic works in the Copais, but cement has now entirely replaced it.

Rock-drilling in the tunnel and at its entrance and exit was done by pneumatic drills, two Ingersoll-Rand 9-inch by 8-inch air-compressors being installed, first at the downstream end and later over the mouth of the tunnel, and driven at their original site by two portable steam engines and at their second site by two caterpillar tractors. A pneumatic sharpener and the necessary furnace were included in the plant.

The labour employed was mostly local, and largely from the villages of Karditsa and Kokkino. About 50 per cent. was female, as female labour in the Copais is more satisfactory than male, and is frequently actually better. There was a shortage of local artisan labour for such a work as the remodelling of the Emissary canal and tunnel, and a camp was constructed at Karditsa to accommodate imported labour.

Tunnel-Deepening.—This was begun in the summer of 1924, the intention being to carry through the deepening on half the width of the tunnel first, partly in order to improve the drainage position and partly to assure the drainage of the bed of the Emissary canal so that work could be started there as soon as possible. After three dry seasons this half-deepening was completed and in 1927 the second half of the deepening was started at the down-stream end. It was during this season that the only serious mishap occurred during the execution of the works. Deepening was proceeding on the left side at 140 metres from the downstream portal, the original tunnel being lined with stone masonry from the downstream portal to just beyond this point. Upstream there were patches of masonry, including short lengths of arch, but the tunnel was mostly bare rock. During the night of the 24th June a slip occurred, blocking the tunnel for a length of about 40 metres. This fall was a serious accident,

as it closed the final outlet of all the waters from the Copais, but by the end of that season all the debris had been removed and the tunnel had been lined to above flood-level. The roof and the hole above it remained a mass of timber for that winter, an inspection platform being run in at about the springing of the arch from the downstream end. In high flood, however, it was impossible to use the platform. All went well that winter, and the fall was repaired and the remaining excavation in the tunnel completed the next summer. The question of lining the whole of the tunnel has frequently been considered since, and it has been decided to do so at the first opportunity. A start will probably be made in 1938.

Emissary Canal.—The work of removing the top-hamper went on steadily from 1923 to 1927. Provision had, quite rightly, been made for a berm 3·0 metres wide on both sides of the canal above flood-level in view of the considerable depth of the cut, which was already 18 metres deep and would now be over 20 metres deep, and for laying back the sides above flood-level to a slope of 1 in $1\frac{1}{2}$, up to which level the sides were pitched with stone at a slope of 1 in 1. Considerable trouble had been experienced in the last 800 metres of the Emissary canal between the Stone bridge and the tunnel with slips on both sides, which sometimes seriously blocked the channel in mid-winter. Counterforts had been built in the sides and drains built behind the spoil-banks with masonry leads into the canal, but the slips continued. It was for this reason that the berms were provided and the sides above cut back. Further, in the deepest part, from the Stone bridge to the tunnel, the dry stone pitching was reinforced by a series of counterforts in cement masonry carried up each side and across the bed of the canal.

Work on the actual deepening of the canal began in 1928, the year the repairs to the fall in the tunnel were finished, and was completed in 1931, by which time all other works had also been finished. The deepening of the upstream end, curiously enough, proved the most difficult, although the depth was the least. It was found that the line of springs leading from Karditsa village, from which the water-supply for the work had been taken, continued underground across this stretch of the Emissary canal, and considerable trouble was experienced with water when deepening this length, pumps being employed to help in keeping the bed free of water. During the 1929 and 1930 dry seasons a plough, drawn by a small caterpillar tractor, was employed to loosen the earth in the bed of the canal. This proved a most efficient labour-saving device. Further upstream, where water was encountered, such ploughing was impossible, but, in the last season, 1931, as the depth of the

canal permitted its use, the No. 10 dragline was used on the excavation.

Interior Works.—The remodelling of the Melas river and canal and the Interior canal, together with the construction of the new Great Central drain, proceeded regularly, the earthwork being mostly done by the two dragline excavators, assisted by hand labour for the excavation above water. The works included the necessary bridges, barrages, regulators, etc., and these were all completed by 1930.

It has been explained that the Melas river, prior to the drainage works, found its way to the Grand *katavothra*. The old bed of this river had been cleared and a short canal, the Bastelica canal, cut, so that some of the river could be by-passed this way if for any reason it was undesirable to send it all down the Melas canal. During the progress of the improvement works the Bastelica canal was enlarged and the old bed of the river was given a further clearance in order to keep the water out of the Melas canal as late as possible at the end of the summer, and so to make the dry season, when work could be done in the bed of the Emissary canal and tunnel, as long as possible.

The works executed and the principal quantities involved in the drainage improvement works, together with the capacities of the exterior drainage works, are summarized in the Appendix (Tables III and IV, pp. 303–4).

Results and Benefits.

Four years have now passed since the completion of the drainage improvement works. Although no really severe exterior flood has occurred during this period, water-levels and gradients all show that the position is very much improved, and that, maintained properly, there is very little danger to be anticipated for the exterior drainage works. As regards the interior, gravitational drainage has been found to operate very well indeed. The only serious matter where calculations appear to have been at fault is in the length of time for which it is possible to interrupt gravitational drainage from the Great Central drain. As mentioned earlier, should this drainage be stopped in late winter, flooding of the lowest land begins within 48 hours. This is by no means a normal contingency, and can be dealt with by the old pumping-stations when it does arise. Actually pumps have been used for only 15 days in the 4 years since the works were completed.

The lowering of the level of the Melas river has had most satisfactory results. Not only has infiltration towards the centre been almost entirely stopped, but the drainage of the land lying between

the river and the rock boundary has been enormously improved. Areas which previously could only be cultivated in late spring or early summer are now regularly seeded in autumn.

The new works have also lent themselves to irrigation operations of the greatest value, which would not have been thought of in the days of pump-drainage. It is now possible in a dry autumn or a dry spring to irrigate large areas, either for seeding in autumn or to counterbalance a low rainfall in spring. Only a pumping-plant out of all proportion to normal requirements could have the same effect. Ordinary irrigations in the summer (for cotton) are also largely facilitated by the new works.

Position To-day.

The total area of the original concession of the Company was 60,000 acres. To-day the position may be summarized as under :—

Sold or ceded to claim-holders	7,500 acres.
Occupied by islands, roads, waterways, etc.	5,500 ,,
Cultivated	41,000 ,,
Fallow and grazing land	6,000 ,,
Undeveloped	negligible.
Total	60,000 acres.

The gross annual value of the produce of the Copais may be put, at to-day's prices, at £400,000, or rather less than £8 per acre, so that from an economic viewpoint the enterprise can be considered a success. The capital outlay involved, including all works executed to date, is nearly £1,400,000, or about £27 per acre, whilst the annual cost of maintenance, excluding expenditure on direct farming but including all other charges, is about £40,000, or 15*s*. per acre. The area considered in the above figures is the total area now in the possession of the Company (that is, 52,500 acres), which includes roads, waterways, etc.

Of the developments carried out since the completion of the improvement-works, the following should be mentioned :—The basin called Division "A" lies to the south of the Marsh canal, and the proper drainage of about 2,000 acres of the lowest-lying land in time for cultivation, if flooded, was impossible, owing to the high level of the Marsh canal. As this basin was held in reserve as a flood-escape basin for the Marsh canal, the risk of flooding was too great to encourage cultivation, and the land had not been cultivated for many years. Complete drainage could only be effected by building a siphon under the Marsh canal and taking advantage of the lower

level of the Interior canal. The scheme carried out, and now in successful operation for two seasons, consisted of dividing the basin into two by a strong bank, and providing sluices at the up-stream end of each half through the bank of the Marsh canal to act as flood-escapes, using one half of the basin for this purpose in alternate years whilst cultivating the other. Return sluices are provided at the downstream end of each basin to let off the flood water as far as possible, whilst for final drainage a siphon has been constructed under the Marsh canal leading to the Interior canal, the siphon having two heads in Division "A," one in the eastern basin and one in the western.

The above work, together with several other development-works on a smaller scale with a similar object, have cleared up the areas which were previously rarely drained sufficiently for cultivation, with the result that to-day there is hardly any land where autumn seeding cannot be undertaken with safety.

It is not proposed to describe the irrigation-works of the estate in detail. The Melas irrigation-system has been mentioned, and irrigation of large areas in the centre can be effected from the Melas river. The flow of the Kefissos river, held by barrages in the Grand and Marsh canals, is a very variable quantity in summer, and more barrages are being constructed and are projected to impound water in these canals to neutralize, as far as possible, the irregularity of this supply.

The security acquired by the new works not only resulted in the fuller development of the land drainage and irrigation, but also encouraged development in other directions. Cultivation became intensified. The example set by the Company, who now farm some 10,000 acres with the aid of the latest mechanical devices, soon began to be followed, and the whole standard of farming is rapidly improving. A small repair workshop was soon provided to look after machinery, etc. This has naturally grown and is to-day quite a valuable adjunct. The growth of the shops, allied to the industrial development of Greece and aided in recent years by necessity, due to the difficulty of importing into the country, has resulted in the reduction of orders for spare parts from abroad to about one-tenth in the past 10 years, in spite of requirements being four or five times as large.

The practical abolition of pumping released some 150 kilowatts of hydraulically developed (and therefore cheap) electric power for most of the year. This is being used for many purposes, such as electric welding in the growing workshops, and also to add to the amenities in the Company's headquarters compound at Aliartos, where a power-distribution system has been installed which provides for electric cooking and heating in the British staff houses and for

heating of offices, etc. Besides being very pleasant and convenient, there is also a distinct annual saving.

Although it has taken some time to accomplish, the prophecy of Monsieur Pochet that Greece was peacefully acquiring a whole province is being fulfilled. Some fifty neighbouring villages are now supported in part or entirely on the Copais, and the health and prosperity of the district have improved beyond bounds. At the moment, the international economic position, with import and monetary restrictions, is slowing down progress, but when such matters return to normal the erstwhile feared Copais should rapidly become one of the most flourishing districts in the country.

The consulting engineers to the Company are Messrs. C. S. Meik and Halcrow, and Messrs. Burstall and Monkhouse. The Author's connexion with the Company started in 1921, when he was appointed an Assistant Engineer, later occupying various positions on the Engineering Staff until appointed Chief Engineer in 1929.

The Paper is accompanied by nine drawings, from some of which Plate 1 and the Figures in the text have been prepared, and by four Tables, two of which are given in the text and two in the following Appendix.

APPENDIX.

Table III.

Tunnel:	
(*a*) Excavation of rock	12,861 cubic metres.
(*b*) Masonry lining	950 ,, ,,
(These figures do not include the work done in the fall in 1927 and 1928.)	
Emissary canal:	
(*a*) Excavation of rock	2,177 cubic metres.
(*b*) Excavation of earth	270,161 ,, ,,
(*c*) Stone pitching	24,882 ,, ,,
(*d*) Stone Bridge foundations strengthened.	
Great Central drain:	
(*a*) Excavation of earth	314,444 cubic metres.
(*b*) Bridge barrages	8
(*c*) Bridges	18
(*d*) Pipes	74
(*e*) Regulator	1
Interior canal:	
(*a*) Excavation of earth	328,205 cubic metres.
(*b*) Bridge barrages	2
(*c*) Bridge	1
(*d*) Regulator	1

Melas river and canal:

(*a*) Excavation of earth	126,138 cubic metres.
(*b*) Bridge barrages remodelled	3
(*c*) Bridges remodelled	2
(*d*) Regulator	1

Bastelica canal:

(*a*) Excavation of earth	65,000 cubic metres.
(*b*) Bridge barrage remodelled	1

The costs of the works, which were carried out within the estimate, were:—

	£	*s.*	*d.*
Plant and machinery	21,431	13	2
Tunnel	11,742	0	0
Emissary canal	48,888	9	4
Great Central drain	12,962	12	5
Interior canal	16,212	18	5
Melas river and canal	3,716	0	0
Bastelica canal	1,329	0	0
Total	116,282	13	4

Table IV.—Capacities of Exterior Drainage-Works, After Completion of Improvement-Works.

Point in canal.	Flood-discharge of river entering canal: cubic metres per second.		Discharge of overflow weir or escape sluice with canal flowing to full capacity: cubic metres per second.		Capacity of canal: cubic metres per second.
Grand canal:					
Kil. 23	Kefissos	150	Grand barrage	30	144
Kil. 20	—		Weir	5	—
Kil. 19	Hercyna	110	Weir	55	150
Kil. 17	—		Weir	5	—
Kil. 12	Ponjia	90	Weir	18	176
Kil. 7	Vrastamites	5	—		—
Kil. 5·7	Siacho	15	—		—
Kil. 0	Lophis	60	—		—
Marsh canal	—		—		182
Kil. 8·5	—		Sluice	40	—
Kil. 6·5	—		Sluice	20	—
Emissary canal	—		—		182
Tunnel	—		—		182

The above capacities were calculated with the water flowing 40 centimetres below the tops of the banks of the Grand and Marsh canals.

Discussion.

Mr. Huntsman.

In introducing his Paper, Mr. Huntsman first showed diagrams llustrating the changes which had occurred during the formation of the Plain from the days before Alexander the Great, when the entre of the Plain was an inlet of the sea, down to the present entury. In connexion with the cost of excavation he pointed out hat the smallest unit-costs were obtained with the largest dredger nd the smallest draglines. He said that, where comparable, the heapest power was straight diesel and diesel-electric; that petrol nachines were more expensive; and that coal-burning steam-owered machines were the most expensive. He then showed a umber of lantern-slides illustrating the work, two of which were eproduced in *Figs. 1* and *2* (facing p. 306).

Mr. Halcrow.

Mr. W. T. Halcrow said that it was of particular interest to The Institution to receive two Papers on works of the same class nd in the same country, but with the difference that Mr. Huntsman's Paper described an entirely new project, whereas Mr. Dean's Paper escribed a scheme carried out about 50 years ago, showed the ffect during subsequent years of the drainage on the land, and xplained the further works which had become necessary and which ad been carried out during the last decade.

Some idea of the magnitude of the Salonika Plain works might be ained from the figures given by the Author on p. 261, from which it ould be seen that 61,000,000 cubic yards of earth had been moved, ver 2,000,000 cubic yards having been excavated in 1 month. The ate of silting in the delta of the river Axios was remarkable, and it ould appear that the decision of the Greek Government to proceed t once with the final diversion was sound policy. Perhaps the uthor would say whether that deduction was correct. On p. 267 it as mentioned that some trouble had been experienced through the ction of flood-water on new embankments causing slipping of the ner face. It was always a matter of some difficulty to know to hat extent artificial protection-work should be carried out on e slopes of flood-embankments. The simplest method of dealing

Mr. Halcrow. with the problem was to encourage the growth of grass, but a few years would have to elapse before it would have sufficien strength to resist the erosion of flood-waters accompanied by strong winds.

Mr. Dean had unfortunately not been able to return from Greec to be present at the reading of his Paper, and Mr. Halcrow had undertaken to show a few lantern-slides, most of which had been prepared from photographs taken by himself on his visits to th Copais. Before showing the lantern-slides, however, he woul mention in connexion with the efforts made by the ancient Minyan to drain the lake, that there was an interesting reference to th works of those people in the Foreign Abstracts published in th Minutes of Proceedings of The Institution of 1893,[1] from whicl he would quote the following :—

> "No sufficient information is yet at hand to speak positivel concerning the Katabothren and the mysterious under ground connection between Lake Kopais and the sea But an artificial tunnel, 2 kilometres in length in th neighbourhood of Larymna, is incomplete and probabl belongs to the efforts made to drain the lake in the tim of Alexander the Great. This tunnel, which has 16 shaft formed for its construction, resembles in all respects th Roman works on Lake Fucinus."

Various attempts, such as the introduction of colouring matte had been made at different times to discover where the water flowing into the *katavothras*, the fissures in the rock, reached the se but, so far as he was aware, no results were obtained.

As described in the Paper, the section and gradient adopted b the French engineers for the encircling Grand canal and the Mars canal had in the main proved satisfactory, and there had been littl silting or scouring. A number of schemes for improving the drainag were considered just prior to and after the War, including one whic involved the driving of a new tunnel through the Karditsa ridg Another took the form of widening the Emissary canal and linin the surface with concrete to reduce the friction loss, and thereb increase the discharge capacity. The scheme finally adopted wa that described in the Paper, and in Mr. Halcrow's opinion it was th right solution of the problem.

[1] E. Curtius. "The Dyke-Constructions of the Minyans." Minutes Proceedings Inst. C.E., vol. cxii (1892–93 Part II), p. 423.

Fig. 1.

UPPER NOTCHED WEIR.

Fig. 2.

LOWER NOTCHED WEIR.

965.
97.

Fig. 3.

LAKE COPAIS: TYPICAL BARRAGE.

Fig. 4.

GRAND KATAVOTHRA.

On pp. 298 and 299 the Author referred to a mishap in the Karditsa tunnel which was really a serious one. Had it been impracticable to clear the tunnel during a brief period in the summer when the water flowing into the Copais basin was used for irrigation, the whole area would once again have become a lake. The tunnel was in limestone rock, and was lined with masonry by the French engineers for about 25 per cent. of its length. The Greek contractor appeared to have carried out the deepening in the lined portion by underpinning according to plan, excavating alternate lengths of about 5 feet, building up, and removing subsequently the intervening lengths, until near the end, when he removed a considerable length of rock from under the masonry without leaving supports. As it happened, at that point there was weight on the lining, and it failed by collapsing from the top left-hand of the arch. The directors of the company asked Mr. Halcrow to visit Greece and advise on remedial measures. Examination of the exposed rock at the fall showed that there was a band of serpentine across the tunnel where the fall took place. A system of temporary timbering was decided upon to allow the tunnel to be used for drainage after the summer, and the roof was rebuilt in the following year. Mr. Halcrow.

Mr. Halcrow then showed a number of lantern-slides illustrating the work described in Mr. Dean's Paper, two of which were reproduced in *Figs. 3* and *4*. When the Karditsa tunnel was cut by the French engineers, the water poured into lake Likeri, the level of which was raised as a result by 35 metres. Between lake Likeri and lake Paralimni a further cut was made, and the water level in lake Paralimni was raised by 22 metres. At the far end there was a tunnel through which the water flowed to the sea. It was taken in that way because there was a possibility of constructing a hydro-electric station at sea-level, using the lakes as reservoirs. That scheme was a hardy annual; it came up about once a year, and had done so for the last 30 years.

Mr. K. J. C. Hill said that he proposed to discuss only one major point in Mr. Huntsman's Paper, namely the problems and arguments which led to the adoption of the project for lake Yenitsa and the western side of the plain, with special reference to the Circulatory canal. Naturally many schemes had been investigated, varying from the conveyance of the northern and western catchment rivers directly into the lake and thence to the sea by the Loudias canal, to the collection of all the same streams into an intercepting canal with discharge to the river Aliakmon. Comparative estimates had been prepared for many adaptations and combinations of those two main schemes. The three main objections to the adoption of any Mr. Hill.

Mr. Hill. scheme for passing western catchment rivers into the lake wer (1) the extremely sharp gradient of the rivers between the foothill and the lake; (ii) the high percentage of silt carried by those river during flood-periods; (iii) the enormous amount of excavatio required to cut a canal from lake to sea of sufficient capacity t carry off the entering flood-waters without periodical and seriou inundation of the reclaimed lake area. The unit-costs of such canal would also have been high, but it was the expense involve in trying to overcome the combination of the first two difficultie which led to the abandonment of such projects.

The solution of the problem was therefore sought in an intercept ing canal capable of passing the western catchment rivers to th Aliakmon, the relatively much smaller streams from the norther catchment being passed into the lake and thence to the sea by mean of a small canal, the Loudias canal. The desiderata governing th location of the intercepting canal, besides the usual constant depth o cut, easy gradient, suitable material, and so on, were: (1) closenes to the foot-hills, in order to economize in the required lengths o training works of entering streams; (2) suitable sites for entr of the intercepted streams, where the velocities of flow could b checked and silt deposited at convenient spots for future clearing (3) a suitable point for discharge to the Aliakmon; (4) avoidanc of villages; (5) avoidance of the many rock outcrops.

As the two most northerly rivers of the western catchment, th Moglenitsa and the Voda, had an estimated combined maximu peak discharge representing about 75 per cent. of the total wester catchment flood run-off, special attention was directed to them wit a view to trying to determine their probable flood-duration perio in order to examine the possibilities of the construction of a balancin reservoir in the plain, such being impossible in the foothills o mountains. By great good fortune, a flood occurred in Marc 1928, estimated at two-thirds of the maximum, and the duratio was established. That information, correlated with similar info mation with regard to other rivers entering the Salonika Plain, le to the adoption of the following flood-cycle for the Moglenits and Voda: unlimited normal flood-discharge, which was taken a equal to half the maximum, followed by a 10-hour rise fro normal to maximum, 1-hour duration of maximum, 5-hour fa from maximum to normal, followed again by unlimited norm flood-discharge.

As the construction of a balancing reservoir near the foot-hil to deal with such a flood was found to be very expensive, recour was had to a natural site, consisting of a large area of waste lan

of marshy character, covered with dense low scrub, reeds and under- Mr. Hill.
growth and situated at a fair distance from the foot-hills. The two flanking embankments were formed by the training embankments of the Moglenitsa and the Voda, and a main embankment of about 5½ kilometres in length connected them, thus forming an impounding basin with a possible ultimate surface area of about 7½ square kilometres and an absorptive capacity of about 15,000,000 cubic metres. New cuts to the old channels of both rivers entering were so designed that flood-waters would be well spread over the basin, the undergrowth in which would check entering velocities of flow, aid trapping of silt and detritus, and so reduce "fetch" of waves as to render pitching of the embankments unnecessary except at a few isolated spots. The outlet-control structure to the gathering basin, described on pp. 253 to 255 of the Paper and shown in *Fig. 1* (facing p. 306), was grooved for stop planks for the future use of water from the gathering basin for irrigation purposes.

By the gathering basin, a maximum flood from the two entering rivers of 1,200 cubic metres per second would be regulated to 900 cubic metres per second, and it had been calculated that about 200 years had to elapse before silt deposition in the basin so impaired its capacity that further head-works would become necessary. It was decided, however, that the main canal below the gathering basin should be excavated to full width and depth in order to ensure consolidation of embankments and effect ultimate economy, rather than to have to undertake successive enlargements as the gathering basin became silted up in future years. Thus, beside the "safety-valve" provided inside the basin by the breaching section, an additional factor of safety was obtained by the absorptive capacity of the main canal itself. The adoption of the scheme for the gathering basin in its present location, combined with a suitable design for the outfall structure from basin to main canal, had enabled a very favourable trace for the latter to be adopted, and there was little likelihood of the embankments having to withstand serious pressure for many years to come. At present they had only to withstand about 1½ metres if the maximum flood were to come down. That safety-factor would of course decrease as the basin filled up.

Mr. Oscar Borer referred to the old drainage system started at Mr. Borer.
least by 2,000 B.C., and said that Theophrastus, in his memorandum "On the Nature of Plants," had mentioned the effect of lake Copais on the severity of the winter, as when the *katavothras* could not take the supply of water that was coming down into the lake and the lake rose, the humidity of the atmosphere increased,

Mr. Borer. and the winters were milder than was the case under dr conditions.

Mr. Borer mentioned that he had also seen other statements whicl claimed that the whole basin was under cultivation under the ruler of Boeotia, at that time the Minyae of Orchomenos, which said great deal for what was done in those days. A system similar to th Minyan was adopted in the Fenlands in England, where the catch ment drains ran around the edge of the Fens, picking up what wate could be picked up and diverting it into the rivers instead of allowin it to flow freely into the Fen areas. He noted that at lake Copai the ancient engineers had made use of the high walls of limeston rock, so that they had had to build an embankment only along on side, and the water had been carried between that embankment an the foot of the hills. It was interesting also to recall that in A.D. 40 Epaminondas, in a public-spirited action, had given 6,000 denari for the purchase of cement, which had been used for the first tim in strengthening the walls, and the walls as repaired at that tim could be seen to-day.

From a study of the intensity of rainfall in the area in question combined with the presence of the steep limestone mountains whicl surrounded the basin, he had realized why the rivers rose so rapidl and seemed to fall off just as quickly, a fact which had been use with so much effect by the designer of the Grand canal and th Marsh canal. When referring on p. 292 to the Grand canal, th Author had stated that very little silting or scour had occurred in th lower channel, but from an examination of the plans it woul appear that the canal seemed to have a number of sharp bends, an Mr. Borer could not see how under normal conditions it would b possible to avoid silting on those bends. It occurred to him tha possibly when the river went into flood it assumed a self-cleansin velocity, so that at the end of the winter period those silt deposit on the bends had been cleared away again. If that were the case he would like to know a little more about the nature of the materia out of which the canal had been excavated, because he did not kno how it was possible to have excavation in those areas and th velocities which were given without scour taking place. Incidentally he noticed by comparing Tables II (p. 293) and IV (p. 304 with Table I (p. 291) that the discharge of the Kefissos and th Lophis had been increased by 50 per cent. and that of the Ponji by 12 per cent., and he wondered whether that was due to furthe information which had been obtained as a result of the investiga tions; or was it due to other works which had been carried out o those rivers ?

Mr. Borer.

A similar problem had faced those responsible in regard to lake Copais as in the Fens, due to the shrinkage and the burning of the peat. In the Fens there was also the difficulty that in the summer everything was so dry that the top surface blew off, but under the semi-tropical conditions of Greece he imagined that the summer conditions must be very difficult indeed. He had already, in the discussion on St. Germans Sluice and Pumping-Station,[1] given some figures showing the shrinkage in the Fenlands in England, which might now be taken to have settled down to a loss of about $\frac{1}{2}$ inch per annum. From some information which he had been able to obtain with regard to the Copais, he found that between the surveys made by the French in 1882 and those made by the Company in 1933 the land in the Copais had settled upwards of $3\frac{1}{2}$ metres; he noticed that the Author had taken an average figure of 2 metres, which suggested a fall of $1\frac{1}{2}$ inch per annum.

In connexion with the deep-level drains put in, Mr. Borer wondered whether he was right in visualizing the area as a series of embanked canals running north and south and able to discharge either to the Melas river or to the Marsh canal, depending upon where there happened to be the lesser amount of flood-water. If the Interior canal, Marsh canal and Melas canal, as well as the numerous drains, were embanked, as it seemed to him that they must be, he would like to know whether the Author could say anything about the troubles due to the movements of the banks and to the seepage through the underlying peat.

On p. 297 the Author referred to the deepening of the bed of the Melas canal and river in order to penetrate through the peat into the silt below, but it was difficult to see how the deepening of the canal could have put matters entirely right, because the success of it must depend on the maximum water-levels. The Author had referred to the flooding which occurred from time to time due to the interruption of the gravitational drainage. The assumption was that the seepage took place then as before, but that the improved discharge of the Emissary canal had made the high water-level of less frequent occurrence. He took it that when the Melas regulators were closed, the flood could then pass back and eventually discharge down through the old Grand *katavothra*. It seemed to him that there would always be trouble on account of the water accumulating between those flood embankments.

He would like to ask for further information with regard to the

[1] R. G. Clark, "St. Germans Sluice and Pumping-Station." Journal Inst. C.E., vol. 2 (1935-36), p. 377. (April, 1936.)

Mr. Borer. flood-relief basin in what was referred to as Division "A." He had no knowledge of the amount of silt which was carried in suspension by the river in flood, but it would be remembered that the area was divided into two, and the water was passed in alternate years on to those two sections. If there was very much silt in suspension, he would imagine that the area would begin to be warped up, although that might be a long process as intensive warping up did not occur on the Washlands of the river Ouse; they had been in use for over 300 years, and he could find no records of any considerable rise. If spilling-over was taking place, he wondered whether it was provided for by low grass-covered banks such as had been found sufficient in the Fens, or whether it had necessitated the use of lifting-gates and concrete aprons.

He would be glad if the Author could also give some information about the details of the stone pitching and counterforts referred to on p. 299 for the lining of the canal, because where there was a steep slope, such as 1 in 1, and a small berm, and then a slope of 1 in $1\frac{1}{2}$ over that again, there would come a time when the amount of water accumulating beneath that stone pitching would cause the whole of it to slide in unless there was some form of under-drainage or unless the material was sufficiently porous.

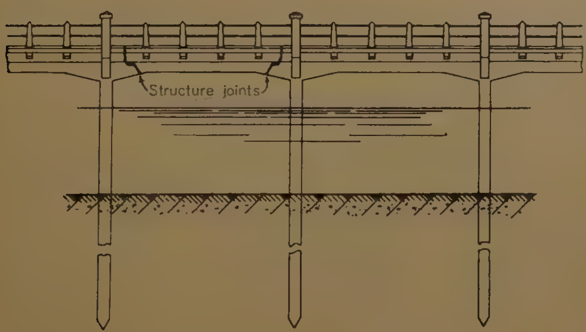
Mr. Turner. Mr. Leslie Turner said that as he had been responsible for 4 years for the design and construction of most of the bridges in the Salonika Plain, he would confine his remarks to a few points arising therefrom. The Salonika Plain consisted of comparatively recent alluvial deposits of unknown depths, chiefly fine silts and sands of high mica-content with occasional ballast pockets. No boring on the bridge sites touched rock, although many of them went down more than 100 feet. As a bearing stratum the ground was everywhere treacherous, and piling was therefore resorted to for all the major structures. The piles were necessarily of the friction type, and experiments were carried out both by driving and withdrawal, and the necessary force to withdraw was measured in order to arrive at and assess a safe friction value. That friction value had been used in addition to the sets worked out in accordance with the formulas for driving, the Hiley formula being generally used.

Another important structural consideration was the very great variations of temperature which occurred in the region, and the high temperatures experienced, which might include prolonged shade temperatures of 110° F. The steel bridges were single spans on rockers and rollers, and presented, therefore, no temperature-difficulties. The longer concrete bridges were constructed on an intermittent system, whereby the value of monolithic construction

was retained, but at the same time ample provision for structural movement was made. That was illustrated in *Fig. 5*, which represented single- or double-connected spans provided with cantilever continuations at the end-bents of piles or columns. A freely-supported span rested on those cantilevers; thus a structure joint was formed passing through the longitudinal beams. The parapets had also to be designed to slide in sleeves at the piers. The horizontal bearing in the structure joint consisted of steel plates separated by copper and covered by lead, thus allowing for both longitudinal movement and for rotation due to deflexion of the centre span. Mr. Turner.

Mr. Turner then showed a number of lantern-slides illustrating typical examples of the bridges, and mentioned that there was

Fig. 5.



approximately 1 mile of single-track railway bridge and 1 mile of road bridge.

Mr. R. S. Cole observed that he had been chief engineer in the Copais some time ago, and he had always been amazed how the ancients had been able to set out long canals without the instruments which were available nowadays. Pansanias spoke of the Melas river and of the Binia and Grand *katavothras*, which were probably due to the dissolving action of water on the limestone rock. When those *katavothras* fell in, the lake formed again. From the several shafts left from Alexander the Great's time, when an attempt had been made to tunnel out towards Larymna, it was possible to see the difference between ancient and modern methods of tunnelling. Judging by the *katavothras*, it would be wise to hurry on with the lining of the Karditsa tunnel in case it should fall in. Mr. Cole.

Mr. J. M. B. Stuart remarked that the question as to whether Mr. Stuart.

Mr. Stuart. rivers running through their deltas should be embanked or not was one which required careful consideration by the engineer. Embankments meant interference with nature's disposal of silt, and although in the Salonika Plain scheme a good deal of provision seemed to have been made for the disposal of silt, he could not help wondering what the silt-position would be there in, say, 50 years' time. In the Irrawaddy Delta in Burma there were nearly 400 miles of Government embankments along the river and its branches, which protected about 1,200,000 acres of agricultural land against floods. There could be no doubt that the embankments had been of immense advantage to the country, but their maintenance was a cause of great anxiety to the engineers who had charge of them. The flood-levels were inclined to rise, and the crests of the embankments had also to be raised to correspond with them from time to time. Changes in the channels of the river were of frequent occurrence, resulting in erosion of the banks and, in extreme cases, in the formation of "cut-offs."

Training or bank-revetment works in a river like the Irrawaddy were not feasible owing to the heavy cost, and the only remedy, when an embankment was threatened by the river, was to construct a retired embankment behind the original one, almost invariably on lower ground than the original embankment and, therefore, more expensive. In some places continued attacks by the river had caused three or four retirements to be constructed before security was attained. It was the opinion of some engineers that the construction of many of the embankments in Burma was premature, and that silt accretion for a longer period should have been permitted before flood spill was cut off from the areas inside the embankments. In the case of the Salonika Plain the demand for agricultural land and the protection of communications had called for a scheme which would have immediate results, and embankments on a large scale had been resorted to. It seemed clear, however, that the large amount of silt which was brought down by the streams was going to cause difficulties in the future.

He would be glad if the Author could give some information as to how the output of draglines mounted on pontoons compared with that of the same machines on the ground. Possibly the reason for the adoption of draglines for the work in question was that they could also be used for excavation on land; otherwise small suction or grab dredgers would seem to be more suitable for excavation under water. It would also be of interest if some information could be given as to the hours per day worked by the mechanical plant and the number of shifts of staff and labourers. From the figures i

Appendix IV (p. 280) it would appear that the large suction dredger Mr. Stuart. and the 6-cubic-yard draglines worked almost continuously throughout each month, and that the other machines worked about the equivalent of 21 24-hour days per month. The total cost of the mechanical plant was apparently included in the total cost of the scheme, namely something like 15,500,000 dollars. The total area benefited amounted to 306,000 acres, so that the capital outlay per acre was about 50 dollars, which might be considered an economic figure. The Author did not give any figure for annual maintenance, but from his remarks on p. 259 of the Paper it seemed likely to be expensive.

Mr. HUNTSMAN, in reply, pointed out that, in connexion with Mr. Mr. Huntsman. Halcrow's question regarding the diversion of the river Axios, only a limited area now remained in the Gulf of Salonika which could be filled in by silt brought down by the river. All that area would be required in future, and the Foundation Company had proposed to fill it in to the best advantage in consecutive sections, commencing near Salonika and moving southwards. The first two diversions would have been short and inexpensive; their embankments would have been widely bell-mouthed and would not have extended all the way down to the sea, and silt would have been deposited on the land during those periods. In that way the danger, mentioned by Mr. Stuart, of the premature construction of embankments in the deltaic area would have been dealt with in the best possible way. It ought to be noted that specially difficult problems would arise in the future owing to the close proximity of the mouth of the final diversion to the outlets of the Loudias canal and the Aliakmon river. The embankments in the Salonika Plain had generally become well-consolidated and their slopes sufficiently grass-covered after two or three seasons, but extensive stone pitching had been necessary in certain sandy localities

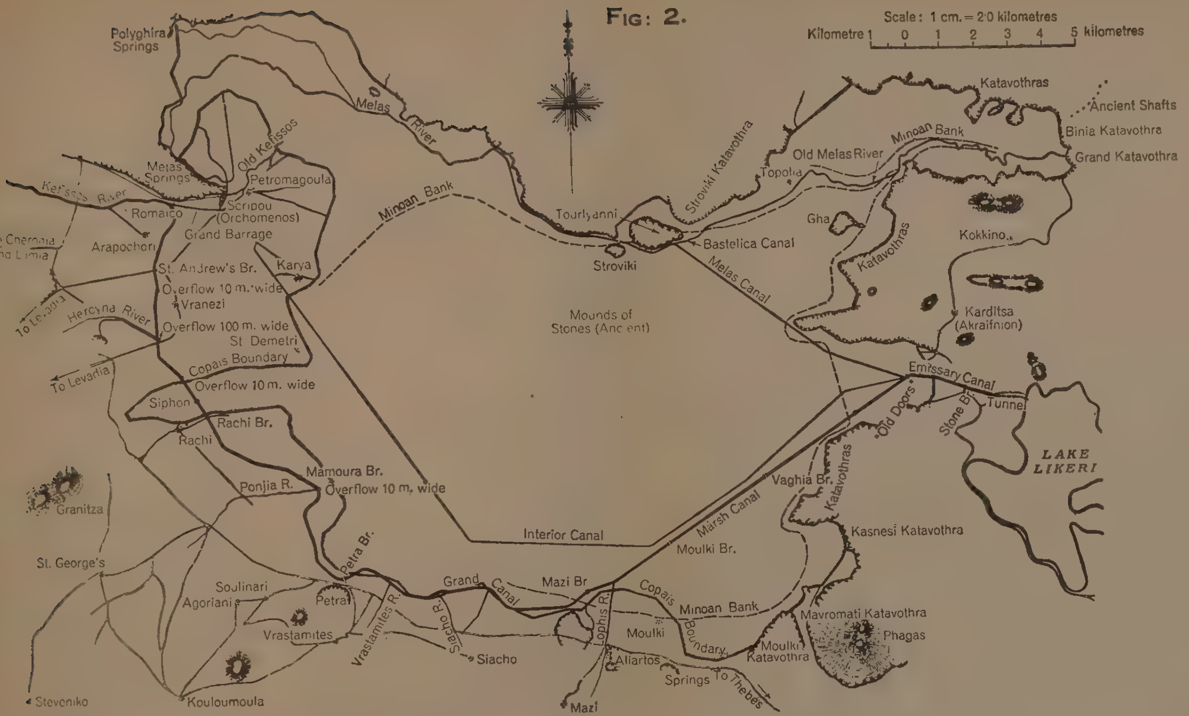
Referring to Mr. Stuart's remarks concerning the excavators mounted on pontoons, Mr. Huntsman said that those machines had given their best outputs when fitted with grab buckets; they had been used in the Loudias canal to excavate the top cut which included the thick bed of reed-roots, and also to form the small bunds used to retain the dredger's spoil. Owing to the loss of material in underwater excavation and to the time taken in moving anchors, similar machines on land would probably have given outputs about 7½ per cent. greater. Three 8-hour shifts were worked on all the machines, with about 24 working days per month, but in the case of the "Axios" dredger, where speed was important, continuous working had at one time been obtained by employing four shifts, each working 9 days on and 3 off.

Mr. Huntsman. The total cost given included the whole cost of all the plant Allowing for the fact that the Greek Government was now employin most of the plant on other works, and remembering that the cos included the diversion of the Axios, works of communication and certain amount of maintenance, it could be said that the cost o reclamation and flood-protection, reckoned on the whole are benefited, was well below 50 dollars per acre.

*** The Correspondence on the foregoing Papers, together witl the Author's reply to the oral discussion on the Paper on "Th Lake Copais, Boeotia, Greece: Its Drainage and Development," will be published in the Institution Journal for October, 1937.—Sec Inst. C.E.

LAKE COPAIS, BOEOTIA; ITS DRAINAGE AND DEVELOPMENT.

FIG: 2.

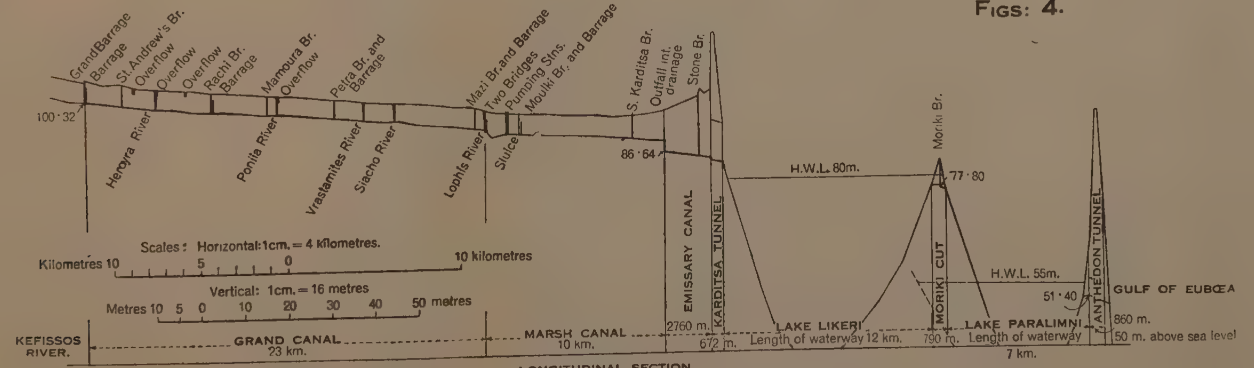


ANCIENT DRAINAGE AND FRENCH PROJECT.

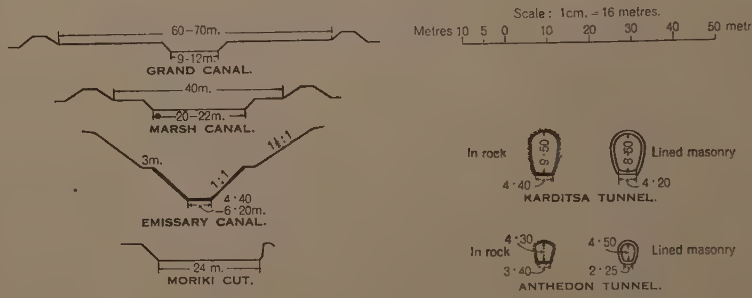
FIG: 3.

PRESENT CONDITION, INDICATING WORKS EXECUTED FROM 1923 TO 1931.

FIGS: 4.



LONGITUDINAL SECTION.



TYPICAL CROSS SECTIONS.

EXTERIOR DRAINAGE-WORKS FROM KEFISSOS RIVER TO THE SEA.

A. J. DEAN.

William Clowes & Sons, Limited: London.

Mr. H

ORDINARY MEETING.

9 February, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

The Scrutineers reported that the following had been duly elected as

Members.

GEORGE FREDERIC BORMAN GOODMAN.

Sir ABDUL HAMID SOLIMAN, *Pacha*, K.B.E.

Associate Members.

RODMELL AGAR, Stud. Inst. C.E.
CHRISTOPHER RICHARD BRODRICK BIRDWOOD, B.A. (*Cantab.*), Stud. Inst. C.E.
ALFRED CECIL BUCK, M.Sc. (Eng.) (*Lond.*).
EDWARD JOHN KNOWLES CHAPMAN, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
FRANK CRAIG DALLIMORE, Stud. Inst. C.E.
KENNETH THOMAS HENRY LANGTON, Stud. Inst. C.E.
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EDWIN LOMAX, Stud. Inst. C.E.
RAYMOND LUND, B.Sc. (Eng.) (*Lond.*).
WALTER GEORGE MCFADYEN.
ELGIN LIONEL MARGO, B.Sc. (*Witwatersrand*), Stud. Inst. C.E.
JOHN HUNT MARSHALL, B.Sc. (*Glas.*).
WILLIAM MATHISON, B.Sc. (*Glas.*), Stud. Inst. C.E.
CHARLES ELLIOTT MOORE, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
JOHN RICHARD PATRICK NORTON, Stud. Inst. C.E.
PETER PAUL ROSENFELD, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
HARRY WALLACE SANDERS, Stud. Inst. C.E.
ALAN EDWARD SHAVE, Stud. Inst. C.E.
ROBERT SLATER, B.Sc. (*Manchester*), Stud. Inst. C.E.
FRED SMART, M.Sc. (*New Zealand*), Stud. Inst. C.E.
THOMAS SPOWART, Stud. Inst. C.E.
JAMES JOSEPH TEESDALE, Stud. Inst. C.E.
LESLIE CHARLES WATERS, Stud. Inst. C.E.
JAN FREDERICK MARCHAND ZOUTENDYK, B.Sc. (*Cape Town*), Stud. Inst. C.E.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5101.

"Fundamental Research on the Application of Vibration to the Pre-Casting of Concrete."[1]

By Donald Arnott Stewart, A.M.I.E.E.

TABLE OF CONTENTS.

Theoretical Considerations.

The function of vibration as applied to the pre-casting of concrete is to effect a redistribution of the particles in such a way that they take up and continue to occupy, no matter for what period the vibration is continued, the most economical position possible. That is to say, all voids must be filled and each individual particle must so interlock with its neighbours that it cannot alter its position without bringing about movement in the whole mass.

The factors which appear to be chiefly concerned in influencing this redistribution are :—

(i.) The action of gravity (vibration is assumed to take place in the line of action of gravity).
(ii.) The neutralization instantaneously of this action.
(iii.) The kinetic energy invested in each particle by virtue of its movement in vibrating.
(iv.) Wedging and packing.
(v.) Adhesion between particles and to the vibrated face.

As the force applied to the concrete alternately assists and opposes the action of gravity, so, instantaneously, its action will be neutralized or augmented in respect to the vibrating system. During the neutral periods the particles are less restrained than at other times and tend to twist and turn much as a stone twists and turns when falling freely. On the other hand, during the alternate periods when the action of

[1] Correspondence on this Paper can be accepted until the 15th July, 1937.—Sec. Inst. C.E.

gravity is augmented, wedging and packing ensue. It may be assumed that the occurrence of redistribution is almost entirely due to these alternating stresses imposed by the forces acting on the concrete.

Great importance is attached by the Author to the effect of the generation of kinetic energy in the particles on account of the segregation it produces. Table I will stress the importance of this aspect of the matter, and at the same time make clear the desirability of giving careful thought to such points as frequency and amplitude when dealing with the vibration of concrete.

Concrete, when in a state of vibration, may be considered to form a kind of colloidal system, since it is a system of relatively large particles suspended in very small particles, and hence a study of the movements of colloidal matter may be of considerable value when dealing with concrete in vibration.

It is worth while to examine a hypothetical case of concrete made up of spheres of radii R_1, R_2, R_3, etc., and subject to sinusoidal vibration. If it is assumed that all particles are of the same substance, the mass of each will be proportional to the cube of its radius. Now the maximum velocity in the cycle will be $A\omega$, where A denotes the amplitude, and $\omega = 2\pi \times$ the frequency.

The kinetic energy will then be $\frac{1}{2}(A\omega)^2(R_1^3, R_2^3, R_3^3, \ldots R_n^3)$.

Hence if R_1 is large as compared with R_n then the kinetic energy invested in a sphere of radius R_1 would enable it to move further in the direction of action of the motive force, after that force has ceased to act, than one of radius R_n, if the resistance to movement were the same in each case. Actually, however, this resistance will depend to some extent on the surface offered by the respective spheres, which is proportional to the square of the radius. Hence the tendency to segregate is directly proportional to R^3, and inversely proportional to R^2. Segregation may therefore be expected to depend on $\frac{1}{2}(A\omega)^2R$.

TABLE I.

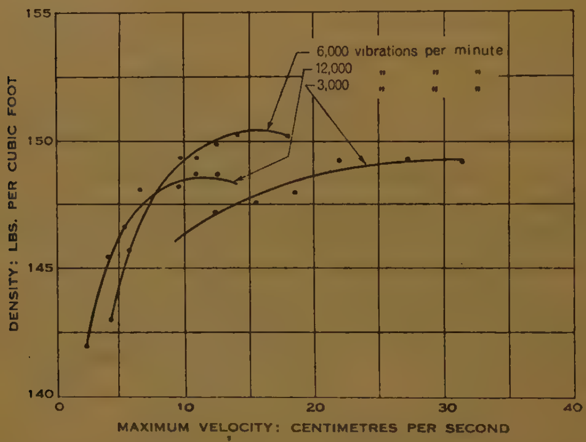
(Maximum acceleration 4·25g,* constant at all frequencies.)

Frequency: cycles per minute . .	1,000	2,000	3,000	4,000	5,000	6,000	7,000	8,000	9,000	10,000
Kinetic energy per gram: gram-centimetres	7,950	2,000	884	500	317	218	163	125	99	80

* Throughout the Paper, g denotes the acceleration due to gravity.

It is evident from these considerations that the best results in regard to consolidation might be expected from the higher frequencies; on the other hand, experiment indicates that a certain amount of kinetic energy (shown by Table I to be greater at the lower frequencies) is desirable in order to assist in the breaking-up of the mix as introduced to the mould and in allowing the free passage of air to the surface. This is made clear in *Fig. 1*, where the relationship between velocity and density obtained with a given mix in the first series of tests has been plotted. It will be noted that density falls off very rapidly below a velocity of about 5 centimetres per

Fig. 1.



second although, as can be seen, the frequency itself still plays an important part.

Passing now to the subject of adhesion, this is almost entirely dependent on water-content, although the grading also has a bearing on the matter. Before the concrete can be effectively vibrated it must have a certain adhesion to the base of the mould. This adhesion can only come about by the removal of air from between the particles adjacent to the base and the base itself. It will be easily seen that, where moisture is lacking or the mix is on the rough side, it is difficult to produce the necessary displacement of air, and consequently the concrete does not follow the mould but vibrates irregularly at a lower frequency and larger amplitude. This phenomenon can only occur where the applied acceleration is greater than that due to gravity. Where, however, a suitable water-content is added to a properly graded mix, adhesion very quickly sets in. No sooner

does this take place in the concrete adjacent to the base of the mould than the water held in its voids is forced upwards by the redistribution and packing which takes place, and adds its quota to the concrete immediately above. The process thus continues until the whole mass is closely following the vibrations of the mould.

Here, as in all matters relating to concrete, water-content is of vital importance. It has been shown that a very dry mix, such as one of the 1-to-2-to-4 type having less than 40 per cent. water/cement ratio, can never consolidate owing to lack of adhesion; on the other hand, a mix of the same nature but hydrated up to 70 per cent. water/cement ratio, will prove equally unsatisfactory inasmuch as the mass behaves as if it were elastic (in the sense in which the term is applied to rubber) and does not vibrate as a whole. Hence in both cases, whether too dry or too wet, the concrete does not follow the movement of the mould but vibrates by occasional impact at a lower frequency and greater amplitude than that of the mould. The consequence is that segregation intervenes in both cases.

The subject of water-content will be raised again in a later section of this Paper.

Description of Apparatus.

The only piece of plant which merits description is that which was used to produce the vibrations. It consisted[1] of a steel platform or table 9 inches by 8 inches by $\frac{5}{8}$ inch, mounted upon a pair of 2-inch by 1-inch by 24-inch channel sections set back to back about 1 inch apart. On the under side of these channels was bolted a bunch of iron laminations forming the armature of a magnetic circuit. The whole of this construction was supported on a pair of bright mild-steel bars 2 inches by $\frac{1}{2}$ inch by 30 inches running at right angles to the table-channels and firmly clamped to them by $\frac{5}{8}$-inch plates and bolts. The free ends of these bright mild steel bars were clamped to the upper flanges of two 6-inch by 3-inch joists of the same length as the bars and running parallel with and below them. These joists in their turn were bolted to a concrete foundation-block weighing approximately 1 ton. Between the joists, and immediately under the bunch of iron laminations, was the magnet, consisting of a group of E stampings with the legs towards the table. About the centre leg was wound a suitably-tapped coil. The magnet was clamped between a pair of 2-inch by 1-inch channels, 15 inches long, running parallel with those of the table, and with their ends bracketed to the web of the foundation-joists. These channels were also secured to the concrete by two 1-inch foundation bolts passing between them and up through a heavy plate, one on either side of the magnet.

[1] A diagram of the vibrating-table is shown on p. 336.—Sec. Inst. C.E.

When the nuts were tightened down, the whole was very firmly fixed and any small movements which might take place due to extension of the bolts could be safely neglected.

The bright steel bars, which were secured to the foundation-joists in such a manner that their length might be varied as desired, acted as springs and were in fact two pairs of double-cantilever springs which, by means of the clamping device, could be tuned to resonate at any load from 60 to 200 lbs. at frequencies varying from 3,000 to 12,000 vibrations per minute. The stresses were kept sufficiently low so that the table might safely be given accelerations up to $20g$.

The method of obtaining vibration was by supplying alternating current to the energizing coil of the magnet, its frequency being half the desired frequency of the table, as there are two peaks of magnetic flux per cycle, one positive and the other negative, both of which attract the armature clamped to the active element of the machine.

Two motor-generator sets were used in order to obtain the necessary range of frequencies, one generating from 25 to 50 and the other from 50 to 100 cycles per second. Frequencies from 3,000 to 12,000 cycles per minute could thus be applied to the concrete.

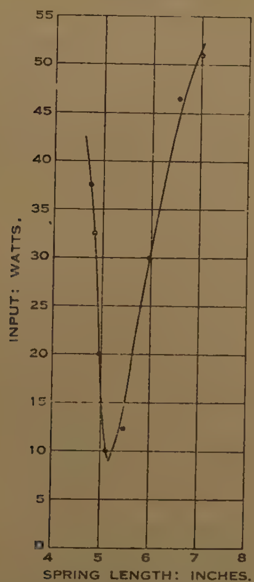
The acceleration of the vibrator was controlled by means of rheostats placed in series with the magnet-coil. By adjusting these rheostats any required voltage could be applied to the coil, and thus any required flux-density could be obtained. Arrangements were made to measure the power-input to the electric circuit, as well as the voltage and current, so that not only could the kilowatt-hours per lb. of concrete consolidated be determined, but the power-factor at which the electrical system operated could also be determined. The power-factor was found to be relatively high, when the characteristics of an equivalent inductive circuit are considered, and during the period of "redistribution and de-aeration" it reached a value of 30 per cent.

Calibration of the vibrator.—In order to calibrate and later to control the vibrator a means had to be devised of measuring the very small high-speed movements. These ranged from a maximum of 1·990 millimetre at 3,000 vibrations per minute to 0·0187 millimetre at 12,000 vibrations per minute as a minimum. It was decided to use an optical lever, and for this purpose a mirror mounted on knife-edges was supported between two V-blocks, one block being secured to the active element and the other to a pillar, the end of which was well embedded in the concrete foundation. For calibration direct current was used to energize the magnet, its pull deflecting the springs. This deflexion was registered on a dial-gauge suitably mounted. At the same time the deflexion of a beam of light reflected

from the mirror on to a screen arranged at a fixed distance from the mirror was also measured. In this way the multiplying factor of the optical lever was found.

In theory it is simple to determine the spring-length which will give resonance for any given frequency and load. In practice, however, particularly when dealing with very small deflexions and relatively high frequencies, some discrepancy arises between theory and practice owing to the fact that none of the component parts, however massive, can be considered completely rigid. They all add their quota to the flexibility of the springs, so that it becomes

Fig. 2.

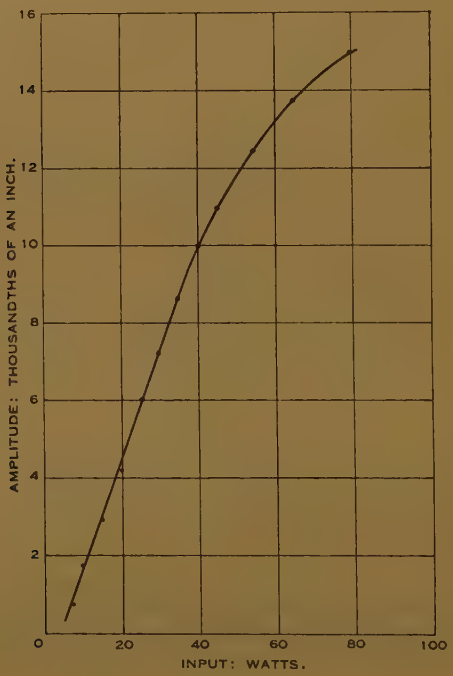


Resonance-Curve with Mass of 110 lbs. at 6,000 Vibrations per Minute.

necessary to make adjustments by trial and error in order to obtain true resonance. For this purpose a mass of 50 lbs. was rigidly clamped to the table, bringing the effective mass to 110 lbs., and the springs were set at such a length that their rigidity was considerably greater than was required. The voltage applied to the coil was then increased until a reasonably large amplitude of vibration was shown upon the screen, and the current, voltage, and power-input noted. The length of the springs was then slightly increased, the amplitude adjusted to correspond with that of the previous reading, and the electrical readings again noted. This process was repeated until

sufficient points were obtained to plot a resonance-curve, as shown in *Fig. 2*. From this experiment it was possible to obtain a correction-factor and, at the same time, to find an exact point of resonance for 6,000 vibrations per minute and 110 lbs. which would allow of a control test. It will be appreciated that the forces acting at exact resonance may be so great as to become dangerous should the machine get out of hand. At the same time, the nearer to exact resonance it is possible to work, the smaller the power-input required

Fig. 3.



to carry out a test. The curve of *Fig. 2* shows that resonance occurs at 5·2 inches. The springs were set to 5·25 inches, and a curve connecting amplitude and watts input at 6,000 vibrations per minute was taken (*Fig. 3*). From this curve it can be seen that the control of the machine is good.

Other apparatus.—There is little to be said about the other apparatus used in these experiments, except that the compression-testing machine was one designed for general use and was not specially adapted to the crushing of concrete specimens. It had no spherical

seatings or ground face-plates, so that in all crushing tests the specimens were bedded between a pair of "Essex board" pads. This, in the Author's opinion, may account for the relatively large percentage-variation which occurs in the ultimate strengths for specimens consolidated at the same amplitude and frequency. This variation is particularly noticeable when compared with the maximum variations for density, most of which are under 1 per cent.

Tests.

The sand and stone used throughout the tests was carefully dried and stored so as to keep the mixes as constant as possible. Both sand and stone were commercial materials. As regards grading and washing, tests showed these materials to be very consistent in density and in voids loosely packed, the voids in the sand being 33·3 per cent. and those in the stone (crushed flint, from $\frac{3}{8}$-inch down) 46·9 per cent., by volume.

The mix for each 4-inch test cube was as follows :—

Stone	3·09 lbs.
Sand	1·875 lb.
Cement	0·825 lb.
Water	190 cubic centimetres (7 per cent. by weight).

The proportions by weight were 1 to 2·28 to 3·74.

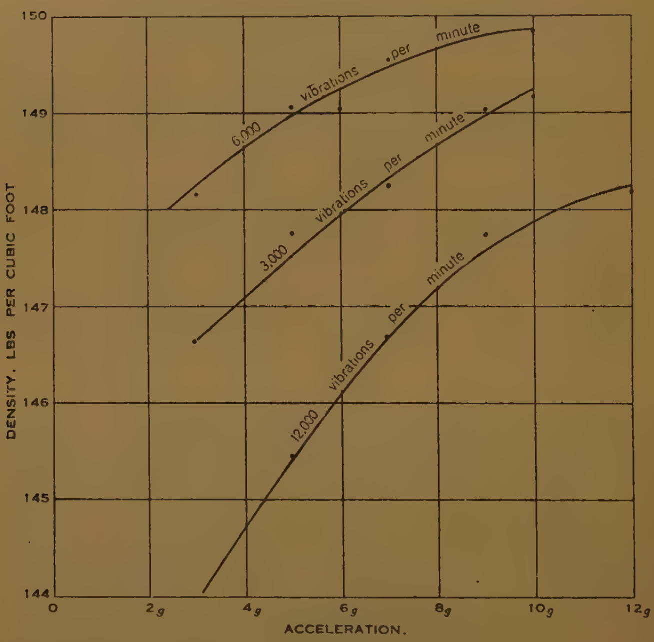
The scientific investigation of the effect of vibration upon concrete has received little or no attention. With only theories and surmises to act as guides some difficulty was experienced in drawing up a programme of tests. It was, however, decided to try to establish a relationship between acceleration and density at a number of frequencies and to be guided by the results of these tests in the arrangement of any future investigation. Time being an important consideration, it was decided that the minimum number of tests should be used per graph-point. A number of preliminary test-specimens were made, and, as these showed surprisingly little divergence from the mean as regards density, it was decided to take the average of four test-cubes per graph-point.

A series of cubes was made at a frequency of 6,000 vibrations per minute and at accelerations ranging from $3g$ to $10g$, each cube being vibrated for 8 minutes. The time of vibration may seem unduly long, but during much of this research the Author had only himself to rely on in the matter of filling and topping up the mould, taking readings of instruments, maintaining the acceleration constant, etc. In these circumstances 8 minutes was only just sufficient, particularly

at the lower amplitudes. As soon as the cubes had been vibrated they were carefully removed from the machine and placed upon the floor of the laboratory. The following day the cubes were removed from the moulds and weighed, after which they were placed in water to cure until required for crushing at 14 days.

Similar tests were carried out at 3,000 and 12,000 vibrations per minute. At the latter frequency the acceleration range was extended to 18g in the hope of gaining an increase in density at the upper

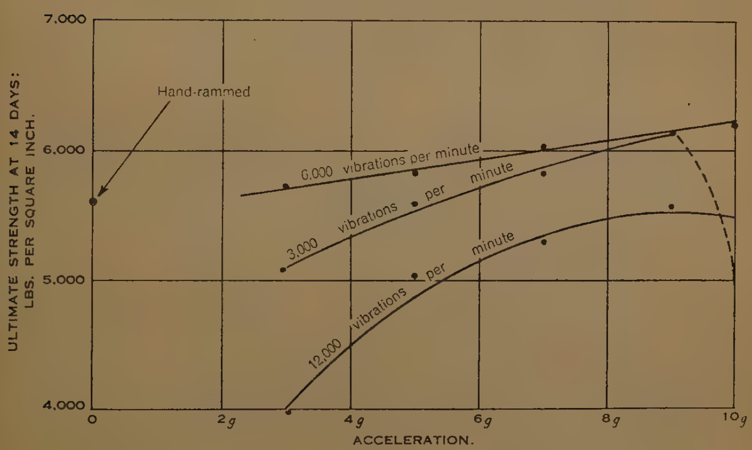
Fig. 4.



limit, but, as the curves for the frequency show, there is no indication of the density or strength ever reaching the figures obtained when operating at 6,000 vibrations per minute (*Figs. 4* and *5*). At an acceleration of about 10g at 3,000 vibrations per minute it was found that results became very inconsistent, and, although twelve cubes were made in an effort to determine wherein lay the disturbing factor, no satisfactory results could be obtained. The dotted line in *Fig. 5* indicates the points where consistent results ceased to be obtainable, and its slope shows the rapidity at which the strength falls off at the large amplitudes.

It will be noted that the curves do not lie one below the other in order of frequency as might be expected from a superficial consideration of the matter. It was on account of this apparent discrepancy, and of the fact that there was no control of temperature or humidity in the laboratory, that a second series of tests was taken. These, however, agreed very closely with the first series, with the exception of those carried out at 6,000 vibrations per minute. Here a considerable change was noticed in the slope of the beginning of the curve, which in the first series was very much steeper than in the second

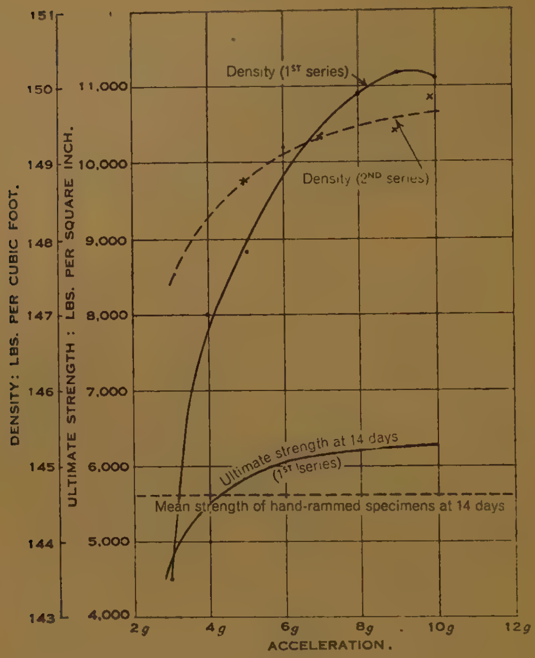
Fig. 5.



(*Fig. 6*, p. 328). This deviation can, no doubt, be accounted for by the fact that during the first test the technique of filling the mould, controlling the amplitude and screeding the specimen was being developed.

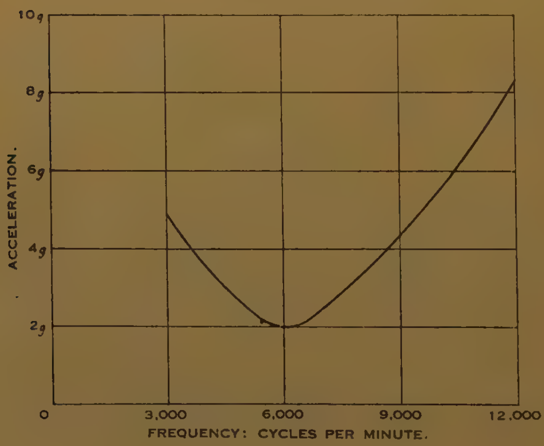
Careful consideration of the probable effect of vibration on concrete led the Author to suppose that, for a given mix, an optimum frequency and acceleration might be found. With this idea in view a set of curves was plotted for a constant strength of 5,500 lbs. per square inch with values of frequency for abscissæ and values of acceleration for ordinates (*Fig. 7*, p. 328). Curves were also plotted of the strength obtained with various accelerations at the frequencies tested (*Fig. 8*, p. 329). These curves show definitely that, for the type of mix and water-content studied, the most satisfactory results might be expected to be obtained at 6,000 vibrations per minute.

Fig. 6.



Density (1ST series)
Density (2ND series)
Ultimate strength at 14 days (1ST series)
Mean strength of hand-rammed specimens at 14 days
DENSITY: LBS. PER CUBIC FOOT.
ULTIMATE STRENGTH : LBS. PER SQUARE INCH.
151
150
149
148
147
146
145
144
143
11,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
2g
4g
6g
8g
10g
12g
ACCELERATION.

Fig. 7.



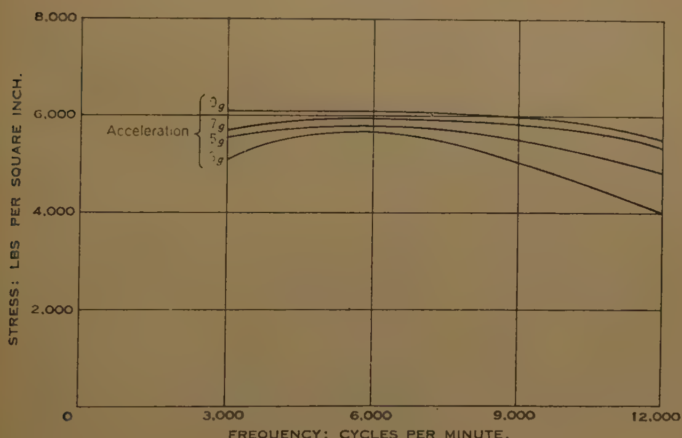
ACCELERATION.
10g
8g
6g
4g
2g
0
3,000
6,000
9,000
12,000
FREQUENCY: CYCLES PER MINUTE.

Phenomena Observed during Tests.

It was seen that the process of consolidation could be divided up into three distinct stages, namely, "initial settlement," "redistribution and de-aeration," and "stabilization."

"Initial settlement" starts as soon as concrete is first thrown into the vibrating mould, or, if the mould is first filled, as soon as the vibrator is started. It consists of a general subsidence of the material, without, however, any change of its general form or of the surface-distribution of the particles. For instance, if the mould is piled high with mix the concrete will sink in the mould in such a way

Fig. 8.



that a mound remains in the centre and the upper corners are left unfilled. During this phase, the duration of which is dependent largely on the water-content (being smaller when the mix is wetter), the concrete does not vibrate at the same frequency and amplitude as the mould, so that, where the mix is too dry, "hunting" can be distinctly heard.

"Redistribution" takes place immediately after the phase just described, the transition from one to the other being very rapid indeed. This second phase of consolidation is very interesting, especially from the point of view of the design of vibrating-machines. It is characterized by a sudden intense activity of the mix. The concrete flows into all corners of the mould, and the surface becomes level and glistens with moisture, whereas previously it was uneven and dull. Major and minor eruptions from which air is ejected

occur at various points all over the surface. The force of ejection is so great at the higher accelerations that small pellets of slurry and sand are shot 3 or 4 feet into the air, accompanied by the hiss of escaping air which can be heard even above the hum of the machine working at 12,000 vibrations per minute and at an acceleration of $18g$. The stones in the mix turn over and over until at last they find satisfactory positions, when they settle down.

The last phase in consolidation, referred to as "stabilization," might be described as a state of complete interlock. The transition from the very active state of "redistribution" is relatively slow. When reached it is characterized by the marked inactivity of the concrete, the surface of which is flooded with a slurry consisting of water, fine sand, and cement. Occasionally a small air-bubble breaks through the surface, but the small amount of redistribution that must have accompanied its ejection is not made apparent by any disturbance of the mix. If the frequency and amplitude are satisfactory, the vibration can be continued without bringing about any further change in the condition of the concrete. Tests have been carried out where the vibration was maintained for 15 minutes after stabilization, and no change throughout that period could be noted in the appearance of the concrete.

The duration of these three stages of consolidation is very susceptible to changes in water-content. A change of 0·25 per cent. will bring about a very marked increase or decrease in the time required to traverse the whole range, particularly when working at the critical water/cement ratio for a given mix. *Fig. 9* clearly shows this.

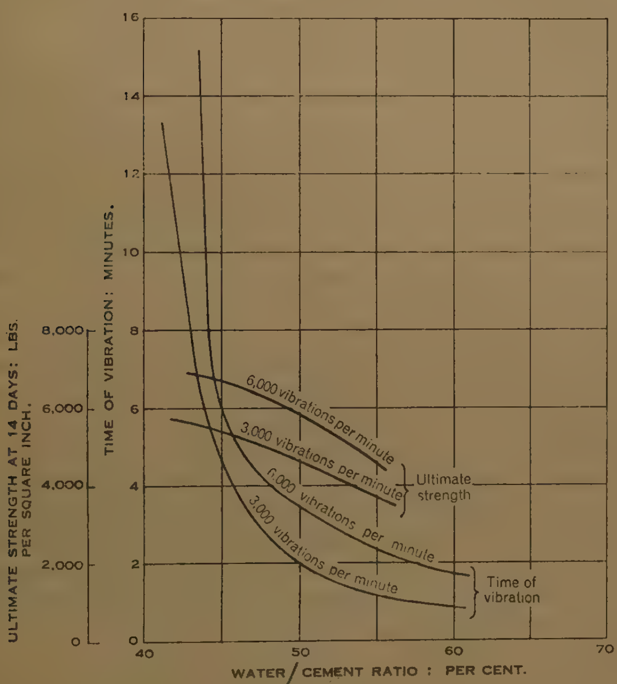
It is interesting to note that, at low frequencies such as 2,000 and 3,000 vibrations per minute, where a large amplitude is used with the object of obtaining rapid consolidation, the third stage is never reached; for short periods amounting only to a few seconds a false stability is attained, but, with the continuation of vibration, the disturbed state of redistribution starts once more. The slurry seems to be sucked downwards from between the stones, leaving them bare; at the same time the concrete gathers itself up in some other part of the mould, leaving a hollow where the uncovered stone is, out of which stones are ejected to dance about the surface of the concrete and finally to fall out of the mould altogether.

When moulds of large area are being filled under vibration it is noticed that the water appears on the surface in patches, which gradually grow in extent until they eventually join and the whole surface becomes uniformly damp. This, the Author finds, is due to local adhesion to the base of the mould, caused possibly by a maldistribution of water throughout the mix, or by the presence,

in that part of the concrete next the bottom of the mould, of a mortar ball which adheres more easily than the surrounding concrete and so forms a vibrating matrix into which the material above embeds itself. Thus consolidation will occur earlier at this point than in other parts of the mould.

The importance of the second stage of consolidation in regard to the design of vibrating plant has already been stressed. It was found that, given a constant power-input, the amplitude of vibration

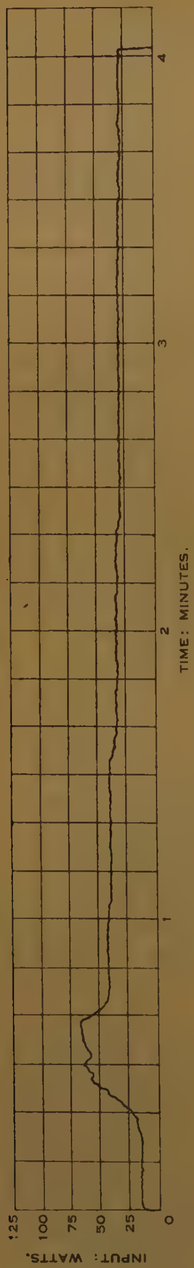
Fig. 9.



fell off sharply during "redistribution" until it reached a definite low limit for a given water-content, after which it rose steadily until stabilization occurred, when it remained constant. On the other hand, it was found that when maintaining a constant amplitude, the power-input increased very appreciably during redistribution (*Fig. 10*, p. 332). As would be expected, the time occupied in consolidation was considerably shorter when the amplitude was kept constant than when the power-input was kept constant.

An analysis of the curves taken on the recording wattmeter (an

Fig. 10.



Typical Record of Power-Input for Constant Amplitude.

example of which is shown in *Fig. 10*) gave the following data: it was found that the ratio of the maximum power-demand to the power required to vibrate the consolidated concrete remained practically constant at all accelerations, the maximum demand being 36 per cent. greater than the final demand in the case of concrete consolidated at 3,000 vibrations per minute and nearly 100 per cent. greater at 6,000 vibrations per minute, whilst at 12,000 vibrations per minute the excess fell off to a little over 18 per cent. as a maximum. It is interesting to bear this in mind when comparing the densities of concrete consolidated at the three frequencies. It was further noted that water-content, whilst markedly affecting the period of vibration required to bring about consolidation, had little or no effect on the maximum demand. It was evident, nevertheless, that the watt-hours per lb. of concrete consolidated increased with the reduction of the water-content, while for the same water-content and acceleration the watt-hours per lb. consolidated tended to decrease with an increase in frequency. It will, however, be necessary to do further research in order to determine their connexion.

Practical Considerations of the Effects of Vibration on Concrete.

In general it was found that the appearance of the concrete was much affected by the acceleration of vibration, apart from the effects of differences of frequency. In all cases the cubes consolidated at the lower accelerations were very much pitted on the faces at right angles to the vibrated face. There is good reason to believe that the pitting

at these surfaces is in excess of that occurring on any plane parallel to them throughout the cube. This condition was observed on breaking open cubes and making a careful examination of them. The probable explanation of this phenomenon is that the heavier material of the concrete tends to move away from the points of greatest intensity of vibration, such as the base and sides of the mould, whose great rigidity lends itself to transmission of the impulses, towards the centre of the mass, thus bringing pressure to bear upon the occluded air and water, which are forced outward from the centre and make their appearance on all faces with the exception of the base, which is subjected to the greatest pressure of any point in the whole system.

A further point of interest that has been noted, although, as yet, not fully investigated, is the decrease in size of the air bubbles with increase in vibration-frequency. This is best observed at the lower accelerations, from $3g$ to $6g$. For instance, comparing cubes of the same density, it is found, generally speaking, that at a frequency of 3,000 vibrations per minute a few large bubbles about $\frac{1}{8}$ inch in diameter are formed, whereas at 12,000 vibrations per minute there are many small bubbles about $\frac{1}{32}$ inch in diameter.

The important bearing that water-content has both upon the strength of the finished product and upon the time required for full vibration has been mentioned. It has also been pointed out that adhesion depends to a considerable extent on the water-content. It may be found that a certain mix requires about 7·5 per cent. of water by weight to allow of the casting being completed in from 2 to 3 minutes. On the other hand, this casting, when fully vibrated, may have a tendency to slump if the forms are removed immediately it has been taken from the vibrator. Under these circumstances it would produce beneficial results if the mix were made in two parts; the first part to be put in the mould should contain, say, from 9 to 10 per cent. of water, while the rest of the mix should be used very dry, having a water-content such that the water-content for the whole casting is reduced to 7 per cent. or even less. Not only will this produce a more stable product, but the face will be free from pitting, whilst the period required for vibration will be reduced by the shortening of the time required to get the original mix to adhere to the base. The excess water from the lower stratum of concrete will very quickly pass up into the mix thrown in on top of it, which will, in turn, reject what it cannot hold. There is no doubt that the most satisfactory way of placing concrete which is to be consolidated by vibration is by allowing a steady trickle to enter the mould. In this way air-pockets are, to a great extent, avoided and the whole process is greatly assisted.

It must always be remembered that a casting which has been made by vibration is a system of particles precisely adjusted, and that this adjustment can be easily disturbed. It is, therefore, important that any product whose strength is to be relied upon should not be subjected to changes of frequency and amplitude after the third stage of vibration has been reached, as otherwise the compacting that has been brought about and the careful grouping of particles that is associated with it, will, in all probability, be disorganized.

One of the claims made on behalf of vibration is that it will allow the immediate stripping of the casting. While this is undoubtedly true in the case of a properly graded and gauged mix, it is by no means so where an undue amount of water is added. This applies particularly to concrete consolidated at frequencies of less than 6,000 vibrations per minute. In spite of the relatively low density of concrete consolidated at 12,000 vibrations per minute, it was found that, on demoulding immediately after vibrating for a period of 8 minutes—approximately 10 minutes after the mix was made—a 4-inch cube would bear the Author's weight and would still remain firm enough to be handled afterwards.

Tests have been carried out with the object of finding what the Author terms the "rigidity-factor" of vibrated concrete, but, so far, the results have not been very reliable. Before proceeding further with these tests, it will be necessary to design and build a special testing machine by which the load can be increased steadily and rapidly and without undue jarring. At the same time, automatic records of distortion and load must be taken. The greatest difficulty in these tests is to avoid the complication due to initial set intervening, particularly where a rapid-hardening cement is used. The following approximate results have been found for concrete of the proportions given on p. 325, with a water-content of 6·5 per cent., vibrated for 6 minutes with a maximum acceleration of $5g$:—

Vibrations per minute.	Breaking load: lbs. per square inch. (10 minutes after first adding water to mix.)
3,000	11·4
6,000	15·4
12,000	16·6

So far, nothing has been said about the relative strengths and densities of vibrated and hand-rammed concrete, the reason being that a true comparison is impossible. In or out of a laboratory, vibrated concrete is more or less the same provided that the technique connected with it is properly understood. With hand-rammed concrete the difference between specimens prepared in the laboratory and those made on a job is very great. For the purposes of this

research it was decided to make four hand-rammed specimens with each series of tests, to act as a check on the cement and also to give a datum with which the strengths of the vibrated specimens might be compared. It was found that the mean strength of hand-rammed cubes having the same water-content as the vibrated cubes was 5,500 lbs. per square inch at 14 days (*Fig. 5*, p. 327), whilst the mean strength of cubes consolidated at 6,000 vibrations per minute was 6,000 lbs. per square inch, an increase of strength of 9·1 per cent. The variation in strength of the hand-rammed cubes was 16 per cent., whereas the maximum variation of any group of four cubes consolidated at the same frequency and acceleration was 10·8 per cent. (for 12,000 vibrations per minute and an acceleration of $4g$).

It is fully realized that the above comparison is open to the criticism of the advocates of vibration, on the point that the low water-contents used in vibration are uneconomical in commercial undertakings where hand-ramming is employed as a means of compacting. Yet, for fundamental research, no useful purpose could be achieved in comparing hand-rammed and vibrated concretes of different water-contents. The Author therefore makes no apology for saying that in regard to strength there is but little difference between the two methods of consolidation. On the other hand, a great improvement is noted in the finish of a vibrated specimen, and considerable advantage will probably accrue from the lower percentage-variation in strength. This latter advantage can only be obtained where both frequency and amplitude are under complete control, which postulates the clamping of the mould, which itself must be properly designed, to the vibrating mechanism.

It is nearly 30 years ago that vibration was first applied to concrete, and yet it is only to-day that elementary principles of this important subject, which holds out such great possibilities, not only of improved products but of larger outputs, have been investigated. Many months, possibly years, may be expected to elapse before the process is sufficiently developed for its value to be fairly assessed.

The Author wishes to express his gratitude to Dr. W. H. Glanville for advice and encouragement during the course of the research, and also to thank Messrs. Associated Portland Cement Manufacturers, Ltd., for supplying Ferrocrete for the tests, and Messrs. Evershed & Vignoles, Ltd., for the loan of the recording watt-meter, from which much valuable information was obtained.

The Paper is accompanied by ten sheets of drawings, from which the Figures in the text have been prepared, and by five photographs and instrument-records.

Discussion.

The Author. The Author showed a cinematograph film of concrete bein consolidated at 3,000 vibrations per minute, and of consolidation a 6,000 vibrations per minute. He also showed a series of lanter slides, one of which was reproduced in *Figs. 11*, and a small han tamper.

He pointed out that, if the input to the table were kept constan a very big movement would occur at first when the concrete wa just being bounced on the table. Then, when adhesion began an the energy demanded by the concrete became greater, the moveme

Figs. 11.

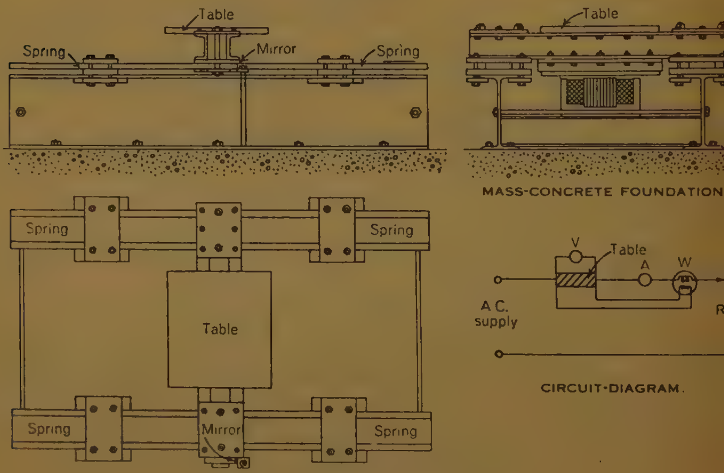


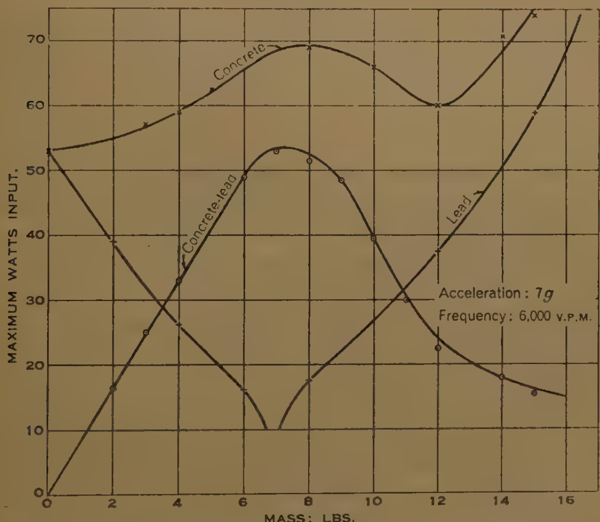
Diagram of Vibrating-Table.

would be less. That could be shown by passing the beam of lig from a mirror on the vibrator through a slot on to sensitized pap wrapped around a revolving drum. The resulting chart showed number of little peaks which were caused by the air being blown o of the mix. The table used for the experiments was very sensiti and therefore any small reaction, such as air being shot out from t mix, was recorded. *Fig. 12* showed that the maximum watts-inp needed to vibrate concrete and lead sheet, given equal masses, we considerably different.

It would be seen that no resonance-point for concrete was obtained corresponding to that for lead of the same mass, that for concrete occurring at approximately twice the mass for lead. The third curve was the difference between input required for the lead and for the concrete. Some misunderstanding had occurred over the use of the word "amplitude." The amplitude was the distance from the datum-line to the crest of the wave representing the motion. The Author.

He had designed a vibrator which could be used by hand. It weighed about 12 lbs., and it was quite effective. He had done some tests on it with concrete of the ordinary 1-to-2-to-4 mix, but using

Fig. 12.



weight instead of volume, with a 5·5-per-cent. water-content. The average strength had been 7,500 lbs. per square inch in compression, and the variation (plus or minus) on the mean had been 2·3 per cent. on four cubes, which had been made by different people.

The PRESIDENT observed that anything that tended to increase the strength of concrete, or to reduce the price, and to make better work, could not fail to be of interest to engineers. The Paper would be of great value to The Institution. The President.

Mr. R. H. H. STANGER remarked that there was one point in the Paper on which he was not quite in accord. On p. 321 the Author spoke of the difficulty of consolidating a concrete of the 1-to-2-to-4 type with less than 40 per cent. water/cement ratio. He had recently been on the Continent, and by means of a vibrating system 1-to-2-to-4 concrete was vibrated with a frequency of from 200 to 300 vibrations Mr. Stanger.

Mr. Stanger. per minute with a water/cement ratio of only 0·35. It seemed to him, therefore, that with further knowledge of vibration and of vibrating machines it might be possible to improve upon present figures for the minimum water/cement ratio. The Research Committee of The Institution had constituted a Joint Committee with the Institution of Structural Engineers to report upon the depositing of concrete by vibrating methods, and the Building Research Station were carrying out the necessary tests for that sub-committee. A special machine had been evolved which was highly sensitive, and he hoped in a short time that a report would be put before the Research Committee.[1]

The Author's division of the process of vibration seemed to be very simple, and if the vibration of concrete were observed it would be seen that there actually were three distinct periods representing initial settlement, redistribution and de-aeration, and stabilization, respectively.

The Building Research Station had been trying for some years to make the British Standards Institution publish a more modern type of specification for Portland cement in Great Britain, and they had been working on the vibrating system for use in the laboratory. If the small hand-tamping machine shown by the Author were made available, laboratory space and expense would be saved.

The Author, on p. 334, referred to the strength of concrete, and mentioned that it was sometimes thought that laboratory tests always gave better results than tests made during the actual placing. That, however, was not the case, and nowadays the works-test could show just as good a result as that made in the laboratory, provided that the same water/cement ratio was used; generally, however, a drier mix was used in the laboratory than at the site.

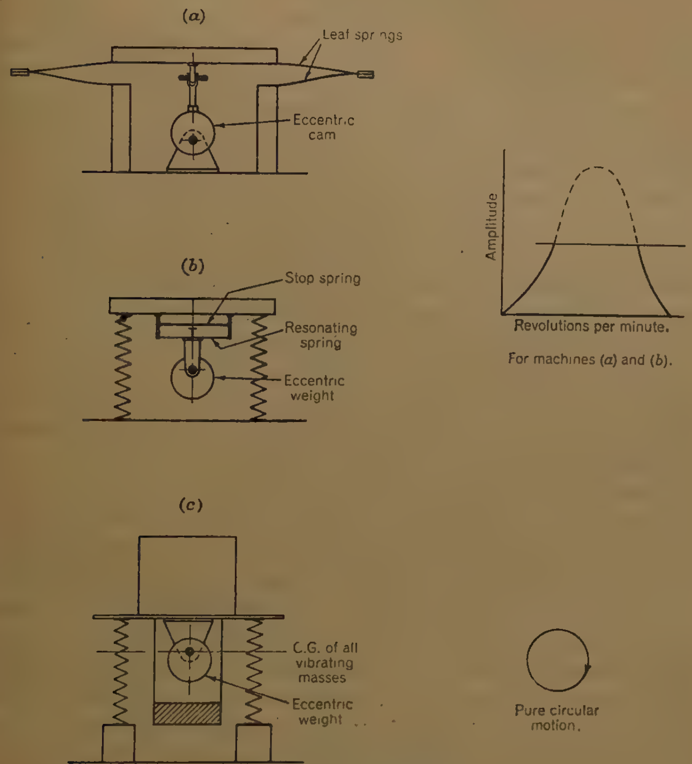
Dr. Glanville. Dr. W. H. Glanville referred to different types of vibrating tables. *Figs. 13* showed three types of vibrating table. That at the top was the Building Research Station experimental table which was being used at present for the investigation of the subject under the joint sub-committee of The Institution and of the Institution of Structural Engineers. The table worked at resonance in the same way as the Author's table, except that the resonance was controlled.[2] The second table was one which had been developed at the Building Research Station in collaboration with a firm of manufacturers of pre-cast concrete by the late Mr. A. J. Newport of the Building Research Station, who also had been responsible for the design of the table at the top of *Figs. 13*. The second

[1] Since published. See p. 435, *post.*—Sec. Inst. C.E.

[2] A description of this table will be found on p. 321.—Sec. Inst. C.E.

table also worked on a resonating principle, but different from the Dr. Glanville. first table. The rotating weight was fastened to a spring, and it was arranged that the weight ran with the natural frequency of the spring. The control of the resonance was obtained by means of a stop spring which prevented the table running away. That type of table, instead of having a purely vertical motion as in the first table,

Figs. 13.



VIBRATING SYSTEMS.

had a very slight lateral motion, but as there was no amplification laterally the motion was very nearly vertical. The third of the tables shown in *Figs. 13* was used for cement-testing and had been developed for testing cement-mortars for the British Standards Specification. The table was arranged so that the centre of gravity of the mass was at the centre of the rotating eccentric weight, thus ensuring that a purely circular motion was given to the concrete. The table had proved very satisfactory for cement-mortars. It could

Dr. Glanville. run at speeds up to 13,000 vibrations per minute and had an acceleration up to 12*g*.

Referring to the research in hand for The Institution, Dr. Glanville showed, as lantern-slides, *Figs. 2* and *4* of the Report of the Joint Sub-Committee on Vibrated Concrete.[1] It would be observed from *Fig. 2* that the best results were shown by the curve for 3,000 vibrations a minute. The Author's optimum conditions were, however, obtained at 6,000, and Dr. Glanville thought that the explanation was that the best conditions of frequency depended upon the water-content and upon the particle-size. The Author's concrete was ⅜-inch aggregate (maximum). The results shown in *Fig. 2* were for ¾-inch aggregate (maximum). Again, Dr. Glanville believed that the water/cement ratio had been 0·49 for the Author's experiments. Both those factors influenced the frequency at which it was best to vibrate. If a high water-content were used it appeared to be better to use a high frequency, and if a smaller particle-size were used it would also be better to use a high frequency. The influence of vibration was not to increase the strength of a particular concrete (provided that that concrete was fully compacted) but was to enable a lower water/cement ratio to be used. On p. 335 was the statement that "The Author therefore makes no apology for saying that in regard to strength there is but little difference between the two methods of consolidation." That was so, provided that there was the same water/cement ratio. Actually, however, the increased strength obtained by using vibration was of the order of 40 or 50 per cent.

Dr. Glanville mentioned other aspects of the research reported on p. 435. He thought that experiments on small tables such as those that the Author had been carrying out would ultimately be of use, not only in the pre-casting of concrete, but in interpreting the results in terms of the actual placing of concrete in situ, so that ultimately it would be possible to specify what sort of agitation should be given to concrete in order to produce satisfactory results.

With regard to the uniformity of results which could be obtained by using vibratory methods, *Fig. 14* showed the remarkable improvement that had followed the use of vibratory methods in cement testing. The maximum variation amongst ten operators from different laboratories using a vibrator was found to be about 3 per cent.

It had been seen that vibration increased the strength of concrete but strength ought not to be regarded as the only feature that mattered. When considering the matter it had to be borne in mind that the greater strength was obtained by reducing the

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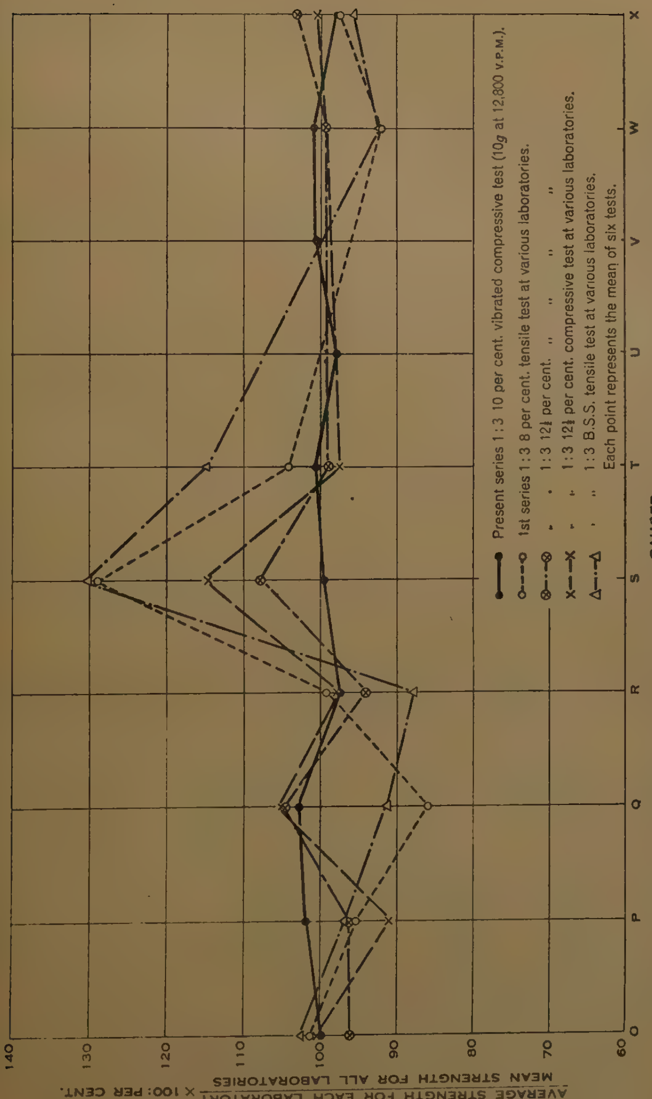
[1] *Post*, pp. 439 and 444.

Dr. Glanville.

Fig. 14.



EFFECT OF VIBRATORY METHODS ON 3-DAY STRENGTHS OF RAPID-HARDENING PORTLAND CEMENT.

Dr. Glanville. cement-content, and at the same time by increasing the aggregate-content and using a very much lower water-content; such a mix could not be expected to have the same properties as concretes mixed with a greater quantity of cement and a greater quantity of water. There were several examples of cases in which troubles had arisen due to that difference. They had only occurred, as far as he knew, in cases where conditions of restraint were present; that was, conditions in which the concrete was unable to move freely and was affected by temperature- and moisture-gradients. In such conditions it appeared that the vibrated concrete was unable to adjust itself in the same way as an ordinary concrete. It was not by any means a general experience, and there were many examples where the difficulties had not occurred, but it had to be borne in mind in using vibratory methods.

Dr Faber. Dr. OSCAR FABER did not understand the Author's description (on p. 318) of the mechanism of vibration in its beneficial effect on concrete, because under (i) the Author assumed the vibration to take place in the line of action of gravity—which meant that sometimes it added to gravity and sometimes it detracted from it, so that it would be possible to get a neutral point when there was no acceleration acting—and under (ii) he assumed the instantaneous neutralization of the action of gravity. The Author then stated that "During the neutral periods the particles are less restrained than at other times and tend to twist and turn much as a stone twists and turns when falling freely." The Author apparently implied that it was the existence of that neutral period, when there was no acceleration acting, which freed the particle to find the best position with the other particles of the concrete. If that were the case it would appear that, during horizontal vibration (such as was caused by most commercial vibrators) that instantaneous neutral period would not be obtained at all; it would therefore be fair to assume that that type of vibration would be quite useless and ineffective. The usual method of vibrating concrete in a beam, for instance, was to clamp an electrical or pneumatic vibrator to the side of the beam; the vibration was undoubtedly horizontal, and did not give the neutral period, but vibration effected in that manner gave excellent results. He thought that the explanation of vibration was that if loose particles were shaken, they would flow in a way different to that in which they flowed if they were left in a static condition. It was probably the reduction due to kinetic friction as compared with static friction.

He did not quite follow the explanation which was given on p. 319 with regard to the tendency for concrete to segregate. The Author stated, quite correctly, that for spheres of the same substance but of different radii, the mass would vary with the cube of the radius,

and he derived an expression for the kinetic energy, but Dr. Faber could not follow the reasoning which led to his conclusion that "... the tendency to segregate is directly proportional to R^3, and inversely proportional to R^2. Segregation may therefore be expected to depend on $\frac{1}{2}(A\omega)^2R$", where R denoted the radius of the sphere. Again, the Author had assumed that there was a constant amplitude A, from which it appeared that the distance which the particle was going to move was settled beforehand. Dr. Faber did not think that it followed that the resistance of the particle which had been set in motion would depend purely on the surface—that was to say, on R^2—as although the pressure which a larger particle met might easily be increased as compared with that for a smaller particle, it did not follow that the resistance would be purely proportional to the square of the radius. He did not, therefore, think that it followed that the segregation would depend on $\frac{1}{2}(A\omega)^2R$. He was not quite clear what the Author meant by "segregation." If he meant the separation of the large particles from the small particles—which Dr. Faber assumed that he meant—the stones for example going to the bottom and the sand coming to the top, that would clearly be something to be avoided. He was not quite clear however, whether the Author was using the word "segregation" in that sense. Dr. Faber.

He asked the Author to explain why (having determined that the segregation was proportional to $\frac{1}{2}(A\omega)^2R$) it followed from that (or from Table I) that the best results from consolidation might be expected from the higher frequencies. Further, the Author immediately afterwards went on to show that experiment proved that with 6,000 vibrations per minute much better results were obtained than were got with either 3,000 or with 12,000. The electrical input was used as a measure of the energy of the vibration, but he was not sure that that was the energy which was put into the concrete, because a certain amount of energy would be wasted in hysteresis losses in the iron inside the core; those losses would undoubtedly be much higher with the high frequencies than they would with the low frequencies, so that the efficiency of the apparatus might easily vary with the frequency.

Apparently *Figs. 4* and *5* (pp. 326 and 327) related to the first series of tests, and *Figs. 6* and *7* (p. 328) to the second series of tests, but as the second series coincided with the first series in certain respects some of the first-series tests appeared to have been introduced in *Fig. 6*, where they were marked "Density (1st series)." That curve appeared to be a reproduction of the corresponding density curve in *Fig. 4* for 6,000 vibrations per minute. *Fig. 7* was based on a cube-strength of 5,500 lbs. per square inch, and it was stated that 6,000 cycles gave the most satisfactory result at an acceleration of $2g$.

Dr. Faber. In *Fig. 6* (which was also based on 6,000 cycles), the 5,500-lbs. figure occurred at approximately $4g$ for the same curve, but *Fig. 7* was supposed to be derived from *Fig. 6*. On p. 330 it was stated that a change of 0·25 per cent. in the water-content would bring about a very marked increase or decrease in the time required to complete the consolidation, and that *Fig. 9* (p. 331) clearly showed that point. In *Fig. 9*, however, the scale of the abscissa varied from 40 per cent. to 70 per cent.; a difference as small as 0·25 per cent. could not be measured on that scale. The Author stated on p. 334 that he was going to carry out some further researches on the "rigidity-factor." What was the "rigidity-factor" to which he referred?

The results given in the Paper would require a good deal of interpretation before the view that they indicated very little difference in strength between vibrated concrete and hand-mixed concrete could possibly be accepted. A great deal of work had been done on vibrated concrete, with results which were approximately twice as good as for hand-rammed concrete; the reason for those good results was that it was possible to use a low water-content. In other words, it made a concrete workable which, without a vibrator, would be entirely unworkable. That, in his opinion, was the real advantage of vibrated concrete, and he thought that merely to compare the strength of vibrated concrete with the strength of hand-packed concrete of the same water-content was ignoring entirely the relative merits of the two methods of treating concrete.

Mr. Durley. Mr. T. C. Durley said that one of the most striking, and evidently one of the most controversial, conclusions to which the Author had come in his researches was that relating to the frequency of vibration. He had been associated, during a period of some 12 months, with the vibration of concrete in situ, and in almost all cases he had vibrated at frequencies approaching 3,000. That had been simply because the manufacturers of the vibrating apparatus had designed it for that frequency and for no other. It was therefore with a considerable amount of relief that he had listened to Dr. Glanville's remarks and had discovered that it was possible to obtain the maximum strength at 3,000 vibrations per minute. It was fairly evident that the explanation of the discrepancy between the results of the Author and of Dr. Glanville was, as the latter had said, due to the different water/cement ratios. That, he thought, rather supported the Author's remarks on p. 319 to which Dr. Faber had taken exception, because it was evident that with the lower water/cement ratio the resistance, which was dependant on R^2, was increased. With a greater amount of water it was reduced, and it was borne out in practice that with a greater amount of water segregation was actually obtained. He had been particularly interested in the figure of $4g$ which Dr. Glanville had given for the acceleration; he had not

een any results for that figure before, but it was the figure that he Mr. Durley.
ad been aiming at for in-situ work. The difficulty was to obtain n adequate amplitude of vibration. Now that that figure had pparently been arrived at he hoped that efforts would be made to liscover a method by which that acceleration could always be btained; it was the most difficult problem in vibration that ngineers had to deal with.

It appeared to him that there was a slight excess of sand in he mix given on p. 325. In practice there was always excess and, but the reason for that excess was because satisfactory mixing vas never obtained. He would be very interested to know in what vay the Author had mixed his concrete and how near he had pproached perfection in his mixing, because if perfect vibration vere possible the mix should also be theoretically perfect, with the ight quantity of sand exactly to fill the voids.

Mr. E. S. ANDREWS remarked that the Paper gave him for the first Mr. Andrews.
ime some definite information as to the importance of the accelera-ions and the frequencies of the vibration. The Author himself ad stated that the Paper was not supposed to be in any way final, ut had pointed out the work which was being done and which had een done up to the present. The difficulty of comparing investi-ations would be reduced if the weight per cubic inch of the vibrated pecimens were measured, because that was surely the test of the ensity of the specimen. He thought that, if there must be strength-ests, they should be made after a longer period than 14 days; further, rovided that only the same aggregates and mixtures for, say, ifferent periodicities and different amplitudes and accelerations vere being compared, then the weight of the specimen should be a ufficient indication of its density.

Mr. H. J. DEANE observed that Dr. Faber had raised the question Mr. Deane.
f the vibration of concrete in a horizontal plane, as compared with he vertical plane used by the Author. He was not disparaging the aluable work the Author had done, nor was he criticizing his method, ut he would suggest that there was a very wide field for investi-ation into the effects of vibration on concrete when it was carried ut in manner which was usually adopted in practice. He assumed hat the watts-input diagram merely indicated the amount of power equired to consolidate the concrete under different conditions of ibration, and so on, in the tests, and that it was not intended in any ay to give the quantity of power necessary in actual practice for ibrating the commercial product.

A further question was that concerning the size and shape of oulds. It seemed to him that there was a good deal to be learned, articularly when vibrating specimens horizontally, about the effect f the shape of the mould on the periodicity and amplitude of the

Mr. Deane.

vibration; that was to say, if a mould containing, say, 1 cubic yar were vibrating, was it necessary to use the same periodicity an amplitude if that cubic yard were a perfect cube or if it were a fla slab? Further, if it were a flat slab, would it make any differenc if the slab were horizontal or vertical?

Dr. Lowe-Brown.

Dr. W. L. Lowe-Brown remarked that the Author had state that vibrated concrete had been used for nearly 30 years, but tha there was very little definite information available which woul enable the benefits which had resulted from vibration to be measure accurately. That might be perfectly true, but anyone who had use vibrated concrete in any form was bound to have been convince that the resulting material was outstandingly better than ordinar concrete.

The Author had given some very accurate information, whic enabled exact conclusions to be drawn for a certain mixture c concrete made with $\frac{3}{8}$-inch material and in 4-inch cubes, but the was as yet no irrefutable evidence that those results were applicab to concrete of other mixtures and sizes, and in larger or small masses. His own experience was that good results could be obtaine with much smaller accelerations than those used by the Author.

No doubt the use of "acceleration" instead of "frequency and amplitude was very convenient to those engaged on researc but it was not easy to visualize; would the Author give the relatio between acceleration and frequency and amplitude? The use "acceleration" had one disadvantage when an attempt was ma to see the effect of varying the frequency while keeping the amplitu constant. For example, unless he was mistaken there were no poin in *Figs. 4* and *5* (pp. 326 and 327) where the same amplitude h been used for any two frequencies. The smallest amplitude on t curve for 3,000 vibrations per minute was larger than the greate amplitude on the 6,000 curve, and similarly there was no comm point on the 6,000 and 12,000 curves.

The Author.

The Author, in reply, agreed with Mr. Stanger's criticism regardi the possibility of consolidating 1-to-2-to-4 concrete with a wat content of considerably less than 0·40 water/cement ratio. T Author had later had an opportunity of experimenting with $\frac{3}{4}$-inch aggregate, and a concrete of that nature had actually be consolidated with a 0·35 water/cement ratio, using an accelerati of $7g$ at 6,000 vibrations per minute.

He was at present investigating the effect of changing the aggrega sizes, and considerable thought had been given to determini whether workability or water-content was the best basis of co parison. It had been decided to carry out a series of tests with t object of finding the corresponding water/cement ratio which wo give the same rate of consolidation for a 1-to-2-to-4 mix containi

maximum stone-size of ⅜-inch, and a similar mix containing a maximum stone size of ¾-inch. Once the right water/cement ratio had been found, the tests already described in the Paper, in which -inch aggregate had been used, would be repeated, using a ¾-inch aggregate. The Author.

The consolidation of concrete in situ by uncontrolled vibration did not come within the scope of the Paper; he suggested, however, that energy-waves propagated from the point at which the vibrator was attached would pass in all directions through the mass of loose concrete confined within the shuttering, and that therefore a vertical component might be expected to act in, or 180 degrees out, of phase with gravity. On the other hand, it was probably true to say that initial settlement occurred due to the reduction of friction between the particles, the reduced friction being brought about by the movement caused by the vibrator. Dr. Faber had, perhaps, taken the hypothetical case of concrete composed of spherical particles too seriously. It was too early to venture upon a mathematical expression of the phenomena associated with vibrated concrete, and the author had introduced that hypothesis simply to act as a stimulant to more fertile minds, and as a possible indication as to the direction along which investigation might proceed, since a somewhat similar expression had been derived by chemists in connexion with coloidal matter. By "segregation" he meant the separation of the concrete into its component parts, which was usually considered undesirable, except as regards the water-content. It therefore followed that, in order to avoid segregation, and at the same time to vibrate at as high an acceleration as possible (*Fig. 5* (p. 327) showed strength increasing with acceleration), it was necessary to increase the frequency so as to reduce the amplitude to within reasonable limits. If the equations connecting acceleration, amplitude, and frequency were examined, it would be seen that amplitude and frequency were related by a square-law:

Let a denote acceleration.
,, F ,, frequency (cycles per second).
,, A ,, amplitude.

$$\text{Then } A = \frac{a}{(2\pi \times F)^2} \quad \ldots \ldots \quad (1)$$

$$\text{But velocity} = aT = a \times 2\pi F$$

$$\text{and therefore } V = a \times 2\pi \times F = A\omega \quad \ldots \quad (2)$$

If the frequency were kept low and the acceleration increased, the kinetic energy would be greatly increased, so that it would be possible, by adjusting those two components, to throw the whole of the concrete from the mould.

Figs. 4 and *5* related to the second series of tests, and *Fig. 6* had

The Author. been put in to show why the development of technique changed th slope of the curve taken at 6,000 vibrations per minute. *Fig. 7* wa derived from *Fig. 5*. The Author regretted that he had not mad his meaning clear in regard to change in water-content. Th statement that " A change of 0·25 per cent. will bring about a ver marked increase or decrease in the time required to traverse th whole range . . ." related to a change of 0·25 per cent. in the weigh of water to dry materials; for instance, from 6·25 per cent. a compared with 6·0 per cent. If the equivalent water/cement rati be calculated for those two water-contents, it would be seen that the were 0·45 and 0·435 respectively. Applying those figures to th curve for 3,000 vibrations per minute shown in *Fig. 9* (p. 331), would be observed that the time required for consolidation wa very considerably affected.

The " rigidity-factor " was the amount of slump or distortio produced per unit-load on a newly-vibrated specimen which wa stripped and subjected to test while still in a plastic state. Th method of arriving at the mix used in the tests had been as follows a mould had been filled with coarse aggregate and vibrated at very low acceleration (about 1·5g) at 6,000 vibrations per minut Dry sand had been added gradually to the coarse aggregate unt the mould had been completely filled. The coarse and fine aggrega had then been separated through a 16-mesh sieve and the weigh had been compared. The results from a number of tests had bee very constant, and it had been found that the mean gave approx mately a 1 : 2 ratio. Cement had been taken arbitrarily as $\frac{1}{6}$ b weight of sand and stone. That mix had been checked on the voi filling system, assuming that the cement had filled the sand-void The whole of the dry materials had been reduced by a percenta on volume equal to the water-volume content at 7 per cent. b weight of dry materials. In practice, as Mr. Durley had pointed ou that arbitrary mix would contain excess sand due to the wetted san bulking when the mixing had been accomplished; therefore t sand-content ought to be, in fact, still further reduced.

The energy-absorption per lb. of concrete consolidated vari very little with the frequency, and was in the neighbourhood 0·0008 kilowatt-hour, which included all losses in the machin Thus, a ton of concrete could be consolidated at a cost of 1·8 unit electricity provided that the water/cement ratio was betwe 0·454 and 0·5. Since the amplitude increased inversely as the squa of the frequency, the point for 12g on the curve for 12,000 vibratio per minute (in *Fig. 4*) was the same as that at 3g on the curve f 6,000 vibrations per minute.

ORDINARY MEETING.

16 February, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

The following Paper was read by the Author, and, on the motion of the President, the thanks of the Institution were accorded to him.

Paper No. 5122.

"Modern Developments in Broadcasting Transmission and Television."

By Sir NOEL ASHBRIDGE, B.Sc., M. Inst. C.E.,
Controller (Engineering), British Broadcasting Corporation.

TABLE OF CONTENTS.

INTRODUCTION.

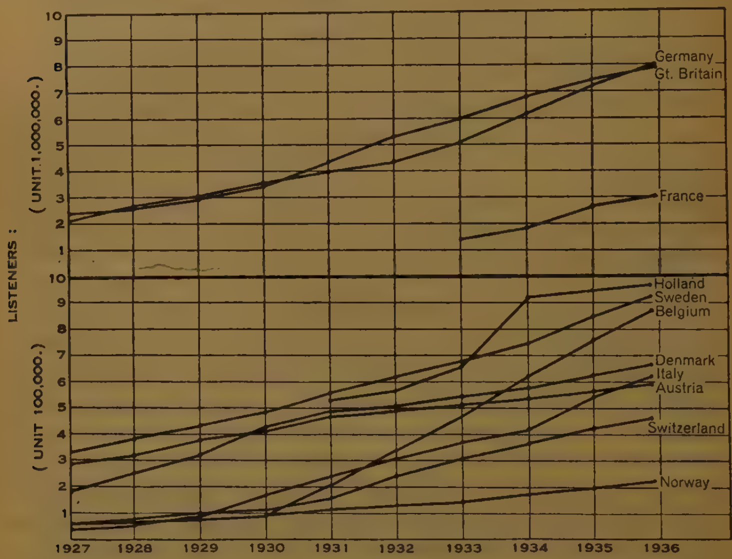
I WISH first of all to express my appreciation of the honour of being asked to speak on modern developments in sound and television broadcasting. For some years past the importance of broadcasting as a potential force in the life of the nation has been generally recognized; I hope, therefore, that this somewhat condensed account of the engineering development during recent years may be of interest, more particularly to engineers who have not been associated in any way with it.

It is necessary to recall the circumstances under which development has taken place. The first regular daily service of broadcasting was established in Great Britain in November, 1922, about 1 year after its introduction in the United States. One or two experimental stations already existed in Great Britain and on the continent of Europe, but it may be fairly said that no recognized technical data existed, and therefore a true estimate of engineering requirements for a service on a national basis was difficult, whilst financial considerations very much limited the scale of early operations.

The first scheme consisted of stations which radiated less than 1 kilowatt, and it was thought at the time that such stations might give satisfactory results up to a range of about 50 miles or more. These stations were afterwards reinforced by a network of even smaller stations radiating 100 watts. These small beginnings,

however, although hopelessly inadequate on a basis of modern ideas, were sufficient to remove doubts as to the financial possibility of establishing a truly national system. Since then development has been extremely rapid, so rapid in fact that there has been at times some difficulty in avoiding dislocation on the engineering side, particularly in connexion with mutual interference from other national broadcasting systems on the continent, which also were growing at a similar alarming rate. *Fig. 1* shows the rate of growth of the number of listeners in Great Britain and abroad during the past 9 years.

Fig. 1.



GROWTH OF NUMBER OF LISTENERS IN EUROPEAN COUNTRIES.

The early difficulties arose partly from a lack of complete knowledge of the particular type of wireless waves in use, and partly from a too slow realization in some countries of the practical effect of the scientific facts as they gradually became elucidated. It may now be said that the latter difficulty has to a large extent disappeared; nevertheless, a complete cure has not yet been found for the ills arising from the operation of an extremely large number of broadcasting stations in a relatively small area.

The broadcasting service in Great Britain may be divided into three distinct categories :—

(1) The home service.
(2) The overseas or Empire service.
(3) Television.

The Home Service.

The home service has as its object the provision of two programmes to the maximum number of persons in England, Scotland, Wales, Northern Ireland, the Isle of Man and the Channel Islands. This has been realized to the extent that, with simple apparatus, 98 per cent. of the population can obtain satisfactory reception of one programme, and 85 per cent. of two programmes.

In considering recent engineering developments in this branch of the service it is convenient to look upon it as comprising two distinct functions, since the technique in each case varies very widely. The first function is to produce a pulsating electric current of low power, the variations of which shall correspond exactly with the pressure of air waves set up by a human voice, or musical instrument, in a broadcasting studio. The second function is to transmit this current so that it may be received by the general public at sufficient strength and constancy so as in effect to exclude all interference, whether of natural or man-made origin. The area over which such reception is possible is usually known as the "service area."

It is proposed to deal with problems of transmission in more detail than those of the studio and microphone, since it is probable that the former will be of more interest to members of The Institution than the latter. Nevertheless, studio problems are of considerable interest, and so an attempt will be made to describe some recent scientific work carried out mainly in the research department of the British Broadcasting Corporation. Certain structural problems are involved in connexion with studio design which are of great importance to the final results obtained, and may, to some extent, apply to buildings not intended for broadcasting purposes.

To appreciate the problem of reproducing music in a manner likely to satisfy discriminating listeners, it must be realized that there is no absolute standard by which engineers can work, the only criterion being musical opinion, which is liable to vary to an alarming extent. When, however, reasonably general agreement has been obtained that results in a particular case are good, it is possible to make measurements which represent the conditions which exist, with some measure of scientific exactitude. Unfortunately, such measurements as are practicable at the present time do not tell quite the whole story.

When a musical instrument is played in a studio at a distance of several feet from a microphone, sound-waves are set up which reach the microphone directly with considerable intensity. At the same time sound-waves are projected outwards towards the walls, ceiling and floor, and normally are in some measure reflected back

to the microphone. Thus the microphone is actuated mainly by waves travelling directly from the source of sound, but partly also by waves reaching it after reflection from the interior surfaces of the studio. The latter will naturally lag somewhat behind the former. It may be said at once that the simple question whether or not a studio can be considered satisfactory is determined largely by the degree and nature of absorption and reflection at the interior surfaces. This may at first sight seem rather obvious, but when sound-waves are reflected by wall- and ceiling-surfaces the following complications may arise :—

(1) Absorption may be selective with regard to the different musical frequencies (extending in normal practice from about 50 to 7,000 periods per second).

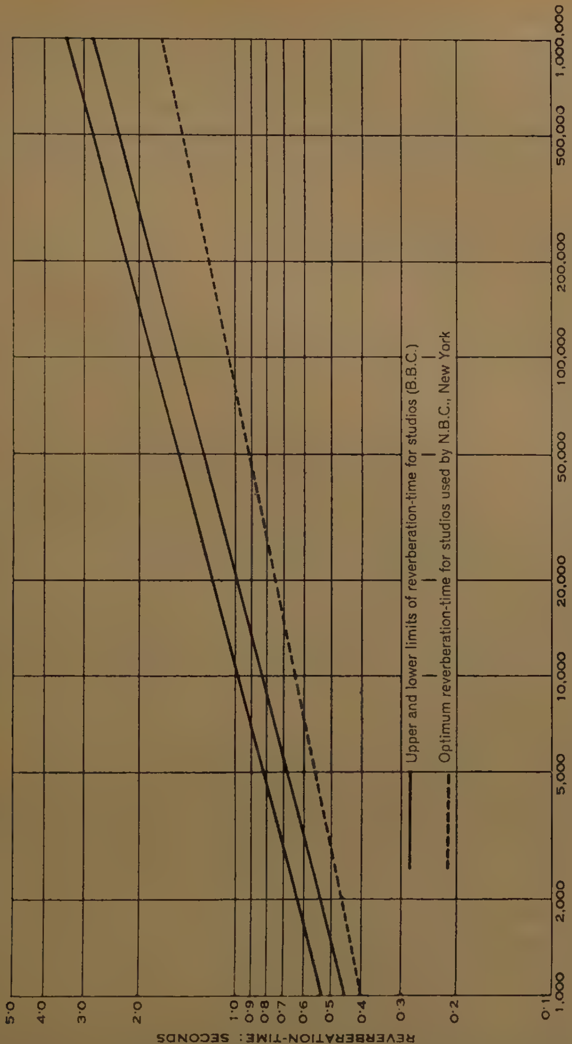
(2) One or more surfaces may not be sufficiently rigid and may act as a flexible diaphragm over a certain band of the musical-frequency spectrum. In general, such an effect is harmful, but it varies in its effect with the degree of mechanical damping possessed by the mass acting as a diaphragm. It is sometimes difficult to visualize a wall or ceiling, which may weigh several tons, moving sufficiently under the influence of the minute pressure impressed by a sound-wave to affect appreciably the quality of broadcast music, but there is ample evidence that such is the case. The effect can, in exceptional cases, be used to advantage to balance some other undesirable acoustic feature, but it is doubtful whether this can be looked upon as an altogether satisfactory basis of design.

(3) The reflected waves may be affected by the shape of the interior surfaces. An obvious case of this would be a curved wall or ceiling giving rise to sound-concentrations. There are, however, important cases where the walls or ceilings, although in general flat, are broken up by heavy pilasters or by deep protruding beams.

The conditions described under (1) and (2) can be examined by taking measurements of what is called the " reverberation time " of the studio. This quantity may be simply defined as the time required for a steady sound of a certain intensity to die down by a fixed amount when suddenly interrupted. Apparatus exists by which such a measurement may be made and automatically recorded. To examine the properties of a hall or studio it is necessary to take a series of such measurements at different musical frequencies ranging usually from about 50 up to 6,000 or 7,000 periods per second.

The first step in the modern method of designing a studio consists of fixing the characteristics of the curve showing reverberation time

Fig. 2.



RELATION BETWEEN REVERBERATION TIME AND STUDIO VOLUME.

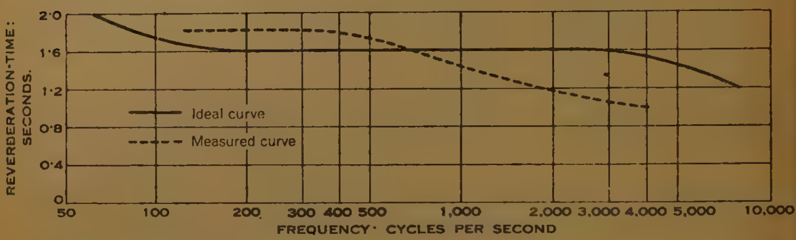
plotted against musical frequency. The various factors which determine this are :—

(1) The volume of the studio.
(2) Whether the studio is to be used for music, or speech, or both.
(3) Whether or not it will be used with an audience.

The curves reproduced in *Fig. 2* show how the optimum mean value of reverberation time varies with studio volume for either

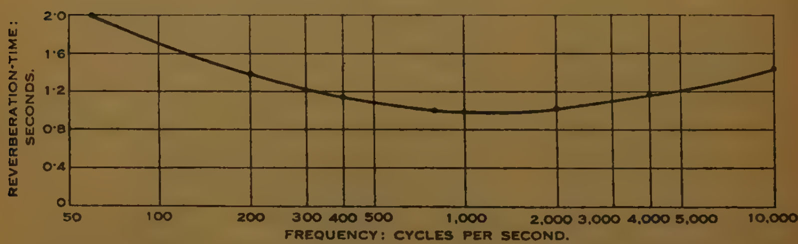
speech or music. When the studio is to be used for both, a compromise becomes necessary. With regard to the third factor, the usual requirement for large studios is that conditions must be correct, either with or without an audience. In order to secure this it has been necessary to find suitable seating accommodation, the sound-absorbing properties of which are sensibly the same whether occupied

Fig. 3.



or unoccupied. It has not been found possible for this to be done with exactitude, but a satisfactory result can be obtained for practical purposes. If a reverberation curve is taken carefully, it shows up many of the defects which a studio may possess, such as the existence of diaphragm effects, or of material used in the treatment of walls which does not absorb sound equally over the range of musical frequencies. *Fig. 3* shows the reverberation curve for the Concert

Fig. 4



Hall in Broadcasting House, plotted alongside what would be considered ideal for this particular type of studio according to our present ideas. The two curves agree as closely as can normally be expected in practice. *Fig. 4* shows the curve for a similar studio according to American practice, which is based on data from an article in the Journal of the Acoustical Society of America.[1] Such curves, however, do not show directly the effect of broken wall- or

[1] Morris and Nixon, "N.B.C. Studio Design." Journal Acoustical Soc. of America, vol. 8, no. 2, October, 1936.

ceiling-surfaces. It is generally agreed amongst broadcasting engineers that curved walls and ceilings are to be avoided as likely to cause sound-concentrations at certain frequencies, thus making the placing of the microphones extremely difficult. The effect of heavy pilasters and the like is uncertain, and it is a somewhat controversial question whether the result is beneficial or otherwise. According to the work in the B.B.C. on this subject there can be no doubt that such features, if pronounced, have a very marked effect on the nature of the reflected sounds which reach the microphone.

An opportunity was recently taken of investigating this effect on a basis of aural comparison, no more satisfactory means being available. It so happened that two studios were to be constructed side by side, which normally would have been identical. In order to investigate this question of irregular wall-surfaces, one studio was designed with plain walls and ceiling, with acoustic treatment designed to give a mean reverberation time in accordance with the volume of the studio and the use to which it was to be put. The other studio was given precisely the same wall- and ceiling-surface treatment, but the walls and ceiling were corrugated, as shown in *Fig. 5* (p. 356), in accordance with a geometric plan intended to prevent any sound being repeatedly returned to the same point as a result of reflection. The measured reverberation curves of the two studios, although not identical, were very nearly so. When, however, the same orchestra plays in each studio in turn under the same microphone conditions the difference between the two, judged aurally, is marked. The impression is given that the studio with the corrugated walls has a shorter reverberation time than the one with plain walls—in particular string instruments seem " dead " and speech considerably clearer. It was found in practice, however, that the studio with the corrugated walls is very satisfactory for military band music, whilst the other is more satisfactory for string orchestras. The exact reason for this is not yet fully understood, although several somewhat undeveloped theories have been advanced. Other work of a similar nature has been carried out to compare the effect of studios with parallel flat walls as against similar studios with converging flat walls. Although it is the invariable practice in some countries on the continent to avoid parallel walls in studios, the experience of the B.B.C. has been that little benefit results from that practice.

At the present time work on studios is principally directed towards the finding of more satisfactory materials for surface treatment, and to an investigation of what is sometimes called the " live end—dead end " principle. The latter consists of arranging the acoustic treatment of a studio in such a way that one end of the studio has surfaces

which are considerably more absorbent than the other. Where this is done, it is usual to place performers in the less absorbent end, and the microphone in the dead end. This practice is not new, but its possible advantages seem to require further investigation in the light of recently acquired information.

It is the practice in some countries to incorporate devices to enable a variation of acoustic conditions to be made at the choice

Fig. 5.

Ground Floor Plan of Studio with Corrugated Walls and Ceiling.

of the engineer or programme producer. The methods used to achieve this naturally vary considerably, and a great deal of ingenuity has been expended on mechanical devices by which a change of acoustic conditions can be made, sometimes merely by operating a switch. The advantages are obvious, especially when it is desired to use the same studio for a large variety of purposes. There are disadvantages, however, of which one is purely a matter of cost. Another arises from the difficulty of ensuring that the moving

structure is what might be termed "acoustically" rigid. It is difficult to predict future tendencies in design, but in Great Britain variable acoustics are not favoured very much. Perhaps the most important conclusion that can be drawn from recent work on this subject is that the acoustics of the studio or concert hall determine to a far greater extent than is generally realized whether a programme sounds pleasing or otherwise to the more discriminating type of listener.

One of the least efficient items of a broadcasting equipment was for many years the microphone, and it is natural that a great deal of research work should have been carried out during the past few years. Various improvised microphones were used in the very early days, but for several years the type in general use depended on the use of carbon in various forms. In this respect the principle still used in all commercial telephone systems was followed. This type of microphone became very highly developed, but its disadvantages gradually became more serious as broadcasting reproduction improved. These were :—

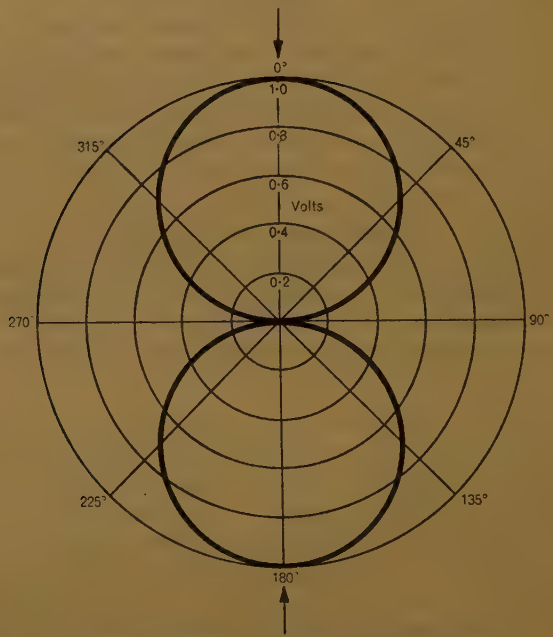
(1) A high degree of spurious electrical noise.
(2) Unequal response over the band of musical frequencies.
(3) Non-proportional response to varying amplitudes of the sound-waves.
(4) Variable performance depending on the direction from which the sounds arrive.

About 2 years ago the B.B.C. developed a microphone of the electro-dynamic type, known as the ribbon microphone. This is now normally used for all studio broadcasting in this country, except in special circumstances. It depends on the movement, set up by the air waves, of a metallic ribbon placed in a powerful magnetic field. The sensitivity of this type of microphone is low compared with that of the carbon type. It is not, however, necessary to house the first amplifier together with the microphone, as is the case with some microphones of low sensitivity, in order to maintain a high ratio of "programme" currents to those caused by spurious induction. This type of microphone varies in design, and is used by many organizations, the advantages being almost the reverse of the disadvantages of the carbon type. They are :—

(1) Low level of spurious electrical noise.
(2) Response over the range of musical frequencies of such a nature as to allow imperfections to be removed by an electrical correction network.
(3) Equal response to air waves of any amplitude met with in practice, that is to say, freedom from overloading.

The B.B.C. design of ribbon microphone does not respond with equal amplitude of output to sounds arriving from every direction, and *Fig. 6* shows the sensitivity plotted in relation to direction of arrival, such a curve being usually referred to as the polar diagram of the microphone. The point, however, which is of obvious importance for the broadcasting of orchestral music, is that there is no discrimination with regard to frequency depending on the direction of arrival of the waves. It can be argued that a polar diagram of the shape shown constitutes an advantage over

Fig. 6.



Polar Diagram of Ribbon Microphone.

one of circular form for most practical purposes. Experiments with a microphone having a curve of the latter form are, however, in progress. In several broadcasting organizations abroad and in many film studios, highly-developed forms of the condenser microphone are still in use, but this type has been abandoned by the B.B.C., chiefly on account of the necessity for a pre-amplifier associated with the microphone itself, and of the general difficulty of maintenance under service conditions.

It is not proposed to attempt a description of the very complicated switching apparatus which is used to control and superimpose the

outputs from a group of studios and other sources of programme material. Such apparatus has been very highly developed to admit of the handling of special programmes consisting frequently, for example, of a combination of programme material coming from countries overseas with speech and music produced in local studios superimposed. The main problem to be solved in such equipment is the elimination of spurious noise caused by the operation of switch-gear and of induction from one circuit to another. It will be realized that the degree of electrical silence which is necessary is not comparable with that permissible in an ordinary commercial telephone system. Studio equipment usually includes a device for producing artificial reverberation. This was originally developed for artificially improving the acoustics of indifferent studios, but it is now entirely used to produce special effects for dramatic purposes. In one particular case, however, the result from a theatre where the orchestra is placed in a pit has been greatly improved by superimposing the reverberation characteristics of a studio especially suited for the type of programme being broadcast.

Most highly-developed broadcasting systems now include in their equipment more or less elaborate apparatus for sound-recording. The B.B.C. uses recording very extensively; the most obvious application is in connexion with the Empire service, where it is necessary to repeat many of the programme items five or six times at different hours of the day. Recording, however, is not confined to the Empire service, considerable use being made of it for illustrating news items and for what are called "feature programmes." This type of programme consists of a vast amount of material collected from many different points and at various times, illustrating the work of a large organization such as Trinity House or a railway system. Time does not permit a description of the recording apparatus used by the B.B.C., but it may be mentioned that there are three distinct methods at present in use. One depends on the use of metal discs coated with a special cellulose material which gives immediate play-back without processing. The second method consists essentially of a moving steel tape on which a magnetic impression of the sound can be made. The third is a mechanical method of recording on a film strip. In connexion with the first method, very highly-developed switching apparatus is used which makes it possible to reproduce two or three selected grooves on a record with precision.

The next link in the broadcasting chain is the telephone-cable between the studio building and the transmitter. Again, it will not be possible to go into this question in any detail, but a brief description of recent developments may be opportune. The conveying of broadcasting material over long distances by telephone-cable is not strictly

a broadcasting problem, being merely a development of telephone technique, but cables suitable for commercial telephone traffic are not suitable, as they stand, for conveying high-quality music. The performance required for broadcasting covers a wider band of frequencies, beginning at about 50 periods per second and going up to a figure between 6,000 and 9,000, as against approximately 200 to 3,000 periods per second for commercial speech. At the same time the tolerable level of spurious electrical noise is only about one-twentieth of that experienced on an average commercial telephone conversation. Lines for broadcasting purposes are provided in Great Britain by the General Post Office and are rented by the B.B.C. Originally they consisted almost entirely of overhead open lines which, although satisfactory from the point of view of frequency response, were subject to a relatively high level of background noise, and to the even more serious disadvantage of unreliability, especially in the winter. A gradual change-over to buried cable has been made and now all studio centres and transmitting stations are linked by these means, in many cases alternative routes being available. Apparatus has been developed to measure and record the performance of all music circuits used. It may be fairly said that the introduction of a 1,000-mile length of cable is not detectable by listening on an ordinary commercial receiver. The total circuit mileage now permanently in use for broadcasting is about 6,000.

We now come to what is probably the most interesting section from the engineer's point of view—the distribution by wireless. To appreciate the problem that exists to-day, it is necessary to touch briefly on past history. When regular broadcasting was inaugurated in 1922, the first stations were allotted such wave-lengths as were not in urgent demand for other services. These were, in the first instance, between 300 and 450 metres approximately, and although broadcasting on such lengths of wave is manifestly a practical proposition, nevertheless, if the question were approached from the purely scientific and economic point of view, wave-lengths as low as 300 metres would not be adopted for stations of 100 kilowatts intended to cover considerable regions. It has, however, become necessary in Europe to use wave-lengths as low as 250 metres, and even below, for this purpose, whilst wave-lengths down to 200 metres are used for local services. It is towards reducing evils arising out of this that a large amount of recent research work has been directed, but it will be necessary to describe the nature of the difficulties before explaining the efforts which have been made to find palliatives.

There is a fundamental fact in connexion with wireless propagation that the longer the wave the less rapidly does attenuation take place as the distance from the transmitting station is increased. Thus,

in average English country, the area covered by two stations of equal power operating on 500 and 250 metres respectively would be roughly of the order of 4 to 1, although this simple proportion does not hold for all cases. If the country is very hilly, the longer wave is even more effective, but there is a limit to the length of wave used, since it is not possible to transmit and receive good broadcasting on wave-lengths above about 2,000 metres if faithful reproduction of music is to be secured. Again, on the very long waves, interference from electrical plant, such as tramways, and the like, becomes intolerable, while the longest aerial which the average listener can erect becomes too short to be efficient. The most suitable wave-band for ordinary broadcasting in Europe would probably be between 500 and 2,000 metres. Unfortunately the wave-bands allocated by international agreement lie between 200 and 545 metres, and between 1261 and 1875 metres. The optimum wave-length for serving the south of England would probably be chosen between 600 and 1,000 metres, but these bands are occupied by highly important ship and aircraft services. It might be thought at first sight that the difficulty could be solved by the use of much greater power on the shorter waves, and that the problem was an economic one. This unfortunately is not the case owing to the existence of what is called reflected or indirect-ray propagation. This is a well-known effect, occurring on broadcasting wave-lengths only at night, which prevents entirely satisfactory reception beyond a certain distance from the station irrespective of its power.

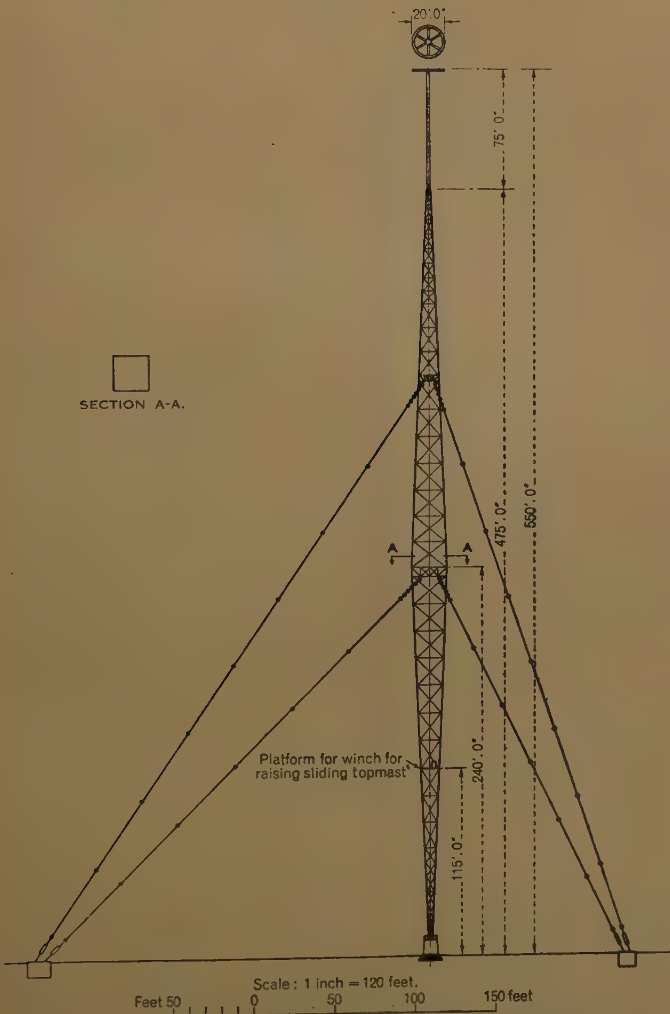
It is necessary to consider this effect in some detail in order to discuss how it has been minimized in recent years. A wireless aerial radiates with varying relative intensity at practically all angles with the horizontal. The horizontal and nearly horizontal components travel over the surface of the earth and attenuation takes place to a degree depending on the topography of the country, on the sub-soil, and on the wave-length used. Radiation at angles above the horizontal travels into space until the waves come in contact with the ionized layers of the atmosphere, now usually referred to as the ionosphere. During the hours of darkness waves of the length used for ordinary broadcasting may be looked upon as being reflected from the ionosphere, with the result that they are re-propagated towards the earth. The point at which any component of such radiation arrives back on the surface of the earth will depend partly on the angle of projection. Since attenuation in space is only a fraction of that occurring along the earth's surface, it will be fairly obvious that certain components of the radiation can travel to the ionosphere and back again to the earth with the same loss of strength as the horizontal component travelling a much shorter distance

along the earth's surface. There will thus be a range of distances from the transmitter over which the reflected wave will be approximately equal to the direct or horizontally-propagated wave. In the area where this condition occurs, distortion, or bad reception, due to phase-differences begins, and continues until the distance is great enough for the direct ray to be small in relation to the reflected ray. At this second point conditions improve somewhat, and whilst, of course, there is no longer a day service because the indirect ray does not exist on these wave-lengths in daylight, fair reception is possible at night by the indirect ray alone. This is the reason why a station 400 miles away, on a wave-length of say 400 metres, gives better reception at night than one of similar design and power 100 or 150 miles away. Moreover, the longer the wave-length, the greater will be the distance at which distortion begins, since the direct radiation attenuates more slowly. It is important to note that raising the power does not alter this distance, since the ratio of direct to indirect radiation remains the same.

A great deal of work has been done in several countries on the design of types of aerial which radiate most strongly along the earth's surface, so that the distance at which distortion takes place due to interference from the indirect ray shall be as great as possible. Such aerials consist essentially of a vertical conductor, the length of which is between 0·5 and 0·6 of the length of the wave used. There are several methods of constructing such radiators, and, although the main theory is generally agreed, the details of design are still somewhat controversial. *Figs.* 7 show in diagrammatic form the design adopted for the 100-kilowatt transmitter at the B.B.C. station at Lisnagarvey, near Lisburn in Northern Ireland. The steel structure of the mast itself forms the conductor for the high-frequency currents. The object of the ring on the top of the mast is two-fold: firstly to enable the physical height of the mast to be reduced somewhat by artificially increasing its electrical length and, secondly, to allow some adjustment to be made in this quantity, either by changing the dimensions of the ring, or, in the case of the Lisnagarvey mast, by raising or lowering the ring. The latter is mounted on a sliding topmast, so designed that it can be raised or lowered from a platform in the main structure 115 feet from the ground. The mast must be insulated from earth; consequently its whole weight of 125 tons, including the vertical component of the stay tension, is taken on an insulator capable in this case of withstanding a test peak high-frequency voltage of 50,000. Obviously, the stays must also be insulated, since they are in effect connected to earth, but the working voltage is much lower in this case, the distribution of potential along the height of the mast being such that it is a minimum

at or near the centre of the mast. In practice, however, it is usual to provide insulation considerably beyond what should normally be necessary, to guard against the uncertain effects of lightning.

Figs. 7.

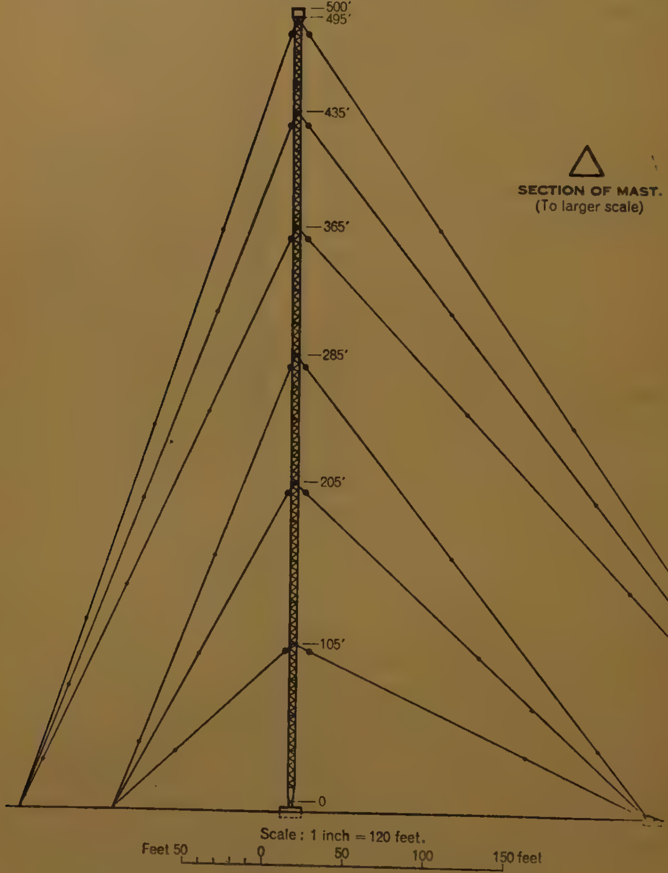


MAST OF TRANSMITTER AT LISNAGARVEY, N. IRELAND.

A number of egg-type porcelain insulators were used in this case, with corona rings to equalize the dielectric strain. This mast has given results which are demonstrably superior to those obtainable with the

older type of aerial consisting of a system of wires supported by two masts. For example, it has made it possible to give a good service in Londonderry, where there would otherwise have been severe distortion and fading (variations of strength).

Figs. 8.



MAST AS USED AT BURGHEAD, N. SCOTLAND.

The type of mast shown in diagrammatic form in *Figs. 8* has a uniform cross-section throughout its length. This is considered to give somewhat improved electrical characteristics. In this design no attempt is made to avoid attaching stays to the mast at points where the electrical potential is high. Such a mast is less costly than the design shown in *Figs. 7* and appears to be at least as

effective, although it may possibly prove to be electrically less robust. The weight, including the vertical component of stay tension, is 68 tons. A mast of this type has been in service for 4 months at Burghead, in the north of Scotland, and so far results have been satisfactory. A somewhat similar mast, 475 feet high, is under construction at the new station at Stagshaw, near Newcastle.

Turning to the design of transmitters, the most outstanding innovations are increased power output and various devices for saving the amount of energy consumed. A transmitter of the ordinary type capable of radiating 100 kilowatts of energy requires a power input of approximately 600 kilowatts, including all auxiliaries. Thus, about 500 kilowatts go to waste in various ways, and it is not easy to use even a part of this energy to heat the building, owing to the necessity of dissipating a large proportion of the waste heat through water at a fairly low maximum temperature. Since most transmitters are in continuous operation for about 15 hours a day, the power cost is considerable, amounting to some £6,000 for each transmitter of this type a year, excluding the cost of stand-by power equipment. There are several well-known power-economizing schemes, three of them being particularly well known. One depends on reducing the amplitude of the high-frequency carrier current in sympathy with the mean level of modulation, and is known as the floating carrier method. The second employs what is known as Class B modulation; the carrier current in this case is kept constant, and economy is effected by reducing the amount of power used for modulation. The third method is somewhat complicated, and cannot well be described briefly. It is known as the de-phasing method. It is of French origin, and is in fairly extensive use in France. With regard to the performance of transmitters in so far as accurate reproduction of music is concerned, comparatively little work has been done in recent years, mainly because distortion is already small and greater amounts take place in other units of the system.

The difficulty of finding sufficient ether space in which all stations in the European region may work without mutual interference has already been briefly referred to, but it is necessary to say something more on this question in order to describe successful attempts which have been made to mitigate this unfortunate state of affairs. If we consider the wave-length plan of stations in Europe in terms of the frequencies corresponding to those wave-lengths, we find that there is, in most cases, a spacing of 9,000 periods per second between adjacent channels. Theoretically this limits the highest musical frequency which can be received entirely free from interference to

4,500 periods per second, but for good-quality broadcasting, frequencies up to about 7,000 periods per second should be reproduced, and up to 10,000 or more periods for the highest possible musical quality. The position, however, is not quite so bad as would appear, since for the most part stations occupying adjacent channels are geographically spaced as far apart as possible, so that the interfering modulation or side-band frequencies of an unwanted station are sufficiently weak in relation to those of the wanted station as to be negligible over a considerable part of the service area. Nevertheless, over the remainder it becomes necessary to limit in the receiver the reproduction of the high frequencies to some value, usually between 4,000 and 5,000 periods per second, in order to exclude interference from stations on neighbouring frequency-channels. At present there is no absolute cure for this state of affairs, which arises from the desire of each country to operate a larger number of stations than can be properly accommodated. It has been proposed to suppress either the whole or a part of the modulation frequencies on one side of the carrier wave. Time will not admit of an examination of the advantages and disadvantages of such a proceeding, but it may be said that it is not as yet generally agreed that such a plan is feasible. It is mentioned here, however, as being the only prospect so far envisaged of being able to produce a wider spectrum of musical frequencies over the whole area served by a station without reducing the number of stations included in the international wave-length plan.

Apart from the question of the limitation of the quality of reception, efforts have also been made to increase the number of stations which can be operated in Europe by the apparently obvious expedient of sharing one wave-length channel between two or more transmitters. When the latter are radiating different programmes this can be effected with little mutual interference provided :—

(1) That the distance between the two stations is of the order of 2,000 miles.

(2) That the carrier waves are maintained at the same frequency with an accuracy of about one part in a million.

It is not a simple matter to arrange that two transmitters situated in countries 2,000 or so miles apart and operated by different organizations shall maintain such a degree of accuracy, but apparatus using piezo-electric crystals, mounted in thermostatically-controlled chambers, has been developed in recent years which makes it a feasible scheme provided that the conditions already mentioned are adhered to, which unfortunately is not always the case. An example of satisfactory working is given by the North Regional transmitter

of the B.B.C. operating on the same channel as the Jerusalem station of the Palestine Government. It does not seem to have been proved that a greater accuracy than about one part in a million improves conditions, so that an extension of this method hardly seems feasible.

There is another method of operating two or more transmitters on a single channel, which has been recently developed to a high degree in Great Britain, and in two or three other countries. This consists of synchronizing the carrier-frequency of two or more transmitters, separated by distances of the order of 150 miles or more, under conditions somewhat analogous to those existing for a number of power stations feeding into a common supply network. With transmitters working in this way at relatively short distances from each other, the same programme must always be radiated, otherwise the reduction of the service area consequent upon the sharing of a channel becomes uneconomic. Moreover, an almost positive locking of the frequencies of the two transmitters appears to give a much improved result compared with that obtained with highly-accurate independent frequency-controlling devices working with a variation of the order of half or a quarter of one part in a million. The method recently developed by the B.B.C. employs a telephone-line between the transmitters to lock their carrier-frequencies together. Recently the same telephone-cable connexion has been used for this purpose as for conveying the programme currents. The frequency of the current transmitted between the stations for this purpose is of the order of 7,500 periods per second, and this is ultimately stepped up to the carrier-frequency of about a million by frequency-doubling amplifiers. This driving-tone current, as it is sometimes called, is separated from the programme currents by carefully designed filters. The less economical but somewhat simpler method has also been used by the B.B.C. of transmitting the locking tone on a separate telephone-cable. In both cases it is necessary to use an electro-mechanical tuning-fork filter in order to prevent traces of spurious electrical current on the line between the stations causing variations of the carrier waves. This method of increasing the number of transmitters without increasing the number of channels used is being extended in Great Britain to improve the percentage of listeners who can receive a second programme. There are, however, two limitations to this method of working; one arises from the fact that distortion is liable to occur in areas where the radiation from the two stations so coupled approaches equality, and the second is that the same programme must always be radiated.

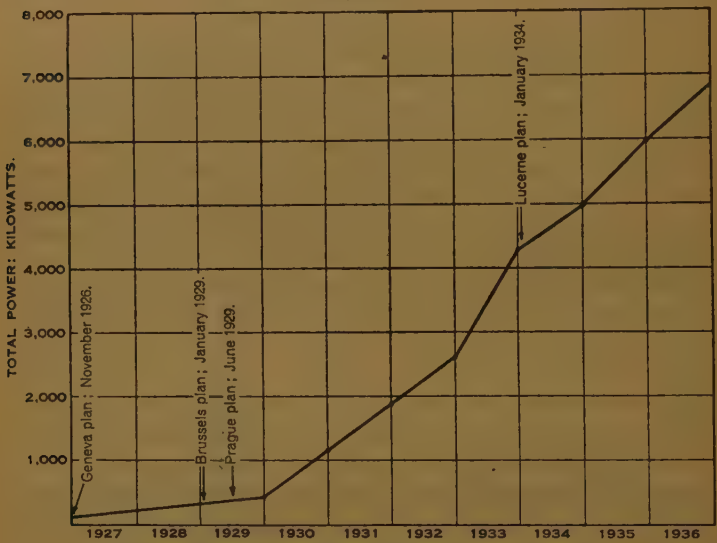
The aggregate aerial power of all transmitters in the B.B.C. home service is now 773 kilowatts, and the units consumed per annum are

approximately 20,000,000. This compares with 37 kilowatts on the 1st January 1926. *Fig. 9* shows the growth of power of transmitting stations in Europe from 1927 to 1936.

The Overseas or Empire Service.

As stated at the beginning of this Paper, the problems of long-distance broadcasting, or what is called in Great Britain Empire broadcasting, are so different from those of the home service that the two are best treated separately. There are, of course, certain problems common to both, such as those appertaining to studios

Fig. 9



European Transmitting Stations, 1927–1936.
Growth of Aggregate Power.

and associated apparatus, and, so far as Great Britain is concerned, the methods used for economizing power. The outstanding difference between the two types of service arises from the type of wave propagation which must be adopted for long distances, that is to say from about 1,000 miles up to the semi-circumference of the earth. It was pointed out that the normal service-range of an ordinary broadcasting station was the maximum distance at which the direct-ray component of the emission was not interfered with by indirect radiation from the station itself. It was stated that beyond this range there existed a zone where reception was particularly bad due to the interaction between the direct and indirect rays at

night. At still greater distances there was, however, a second-class service during darkness by means of the indirect ray only, but ultimately attenuation became too great to give a satisfactory ratio between unwanted and wanted emissions. It is an extension of this latter method of propagation on which very long distance broadcasting depends. Direct-ray transmission for broadcasting is out of the question for long distances on any wave-length without the use of quite impracticable amounts of power. Moreover, ordinary broadcasting wave-lengths cannot be used for the Empire service for two reasons :—

(1) Because indirect-ray transmission on such wave-lengths is not possible in daylight.
(2) Because even in darkness the amount of power required to obtain a satisfactory ratio of wanted signal to atmospheric electric disturbances would be excessive.

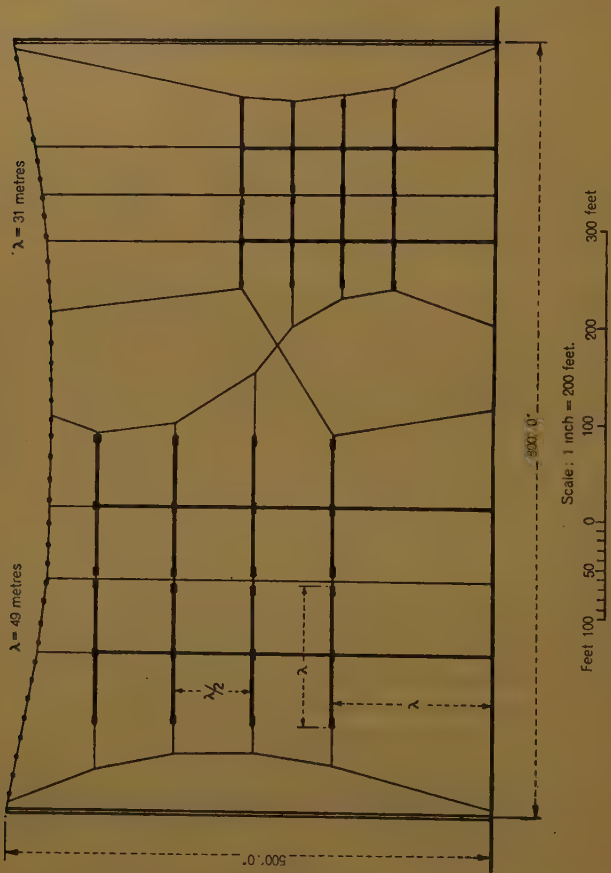
Neither of these objections applies to suitably chosen waves of much shorter length. It is the exact choice of wave-lengths and the design of aerials to use with them which has provided great scope for research during the past several years, but recently some degree of finality has apparently been approached. The particular aim of the B.B.C. short-wave service is to provide reception in every part of the Empire for 2 hours or more during the evening at the receiving end. Consequently the service consists of a series of separate transmissions spaced around the 24 hours, each directed to a particular zone. When the service was opened 4 years ago, although encouraging results were obtained, this general aim was very far from being completely fulfilled, partly because insufficient data were available as to suitable aerial design for this particular purpose. Again, although a considerable amount of data existed as to the various optimum wave-lengths to be used in various circumstances for transmitting over long distances, no data existed which entirely covered the particular requirements of this service. It should be explained that the wave-length for any particular transmission zone may vary from approximately 13 metres to 50 metres, depending on the following factors :—

(1) Conditions of light and darkness over the path of transmission.
(2) The geographical position of the region to be reached.
(3) The season of the year.
(4) The position in a solar cycle of 11 years affecting conditions of reflection from the ionosphere.

The method adopted for determining precisely the optimum wave-length of each zone was the only one available, namely the

organizing of reports from professional and non-professional observers in all parts of the world. For this purpose special log-sheets were prepared on which results could be plotted, following a method laid down for assessing the merit of each transmission. A large number of these blank sheets have been printed and distributed to suitable and willing observers. During the past 3½ years twenty thousand

Fig. 10.



SHORT-WAVE AERIAL ARRAYS AT DAVENTRY.

completed sheets have been returned, from which it has been possible to prepare a schedule of wave-lengths covering practically all conditions occurring in practice.

With regard to aerial design, it may be said that, apart from choice of wave-lengths, this is certainly the most important factor in securing satisfactory results. The type of aerial used is entirely different from that described in connexion with the home service.

Instead of a single vertical conductor, a number of wires suspended in the form of a curtain between two masts is necessary for these short wave-lengths. *Fig. 10* shows a typical aerial bay of this type in which two separate aerial curtains are suspended between two 500-foot masts. It is the design of this curtain of wires which has been the subject of research for several years by many organizations interested in short-wave transmission of all kinds. Since the practice of the B.B.C is to transmit to different zones in turn (there are at present six separate transmissions), it is in general wasteful to transmit in all directions simultaneously. Thus all aerial arrays are arranged so that the energy radiated is projected in the required direction, the width of the beam being just sufficient to cover the distant territory to be served. It is a reasonably simple matter to calculate the dimensions of an aerial required to achieve this, but in practice the results are seldom precisely in a line with the calculations, and some additional data must be obtained from practical observations.

Most of the aerial arrays at present in use at the Empire Station at Daventry send out two beams in opposite directions. The obvious course would be to use reflectors to confine radiation to one direction. So far this has only been adopted for certain aerial arrays because it was found that listeners living in some areas reached by what might be called the " reverse radiation " made use of the service for reception in the morning, particularly for receiving news bulletins (the normal radiation in the other direction being, of course, intended for evening reception). In some cases, however, it has recently been found desirable to use reflectors to prevent what is known as " echo " effect. This is caused by waves reaching a receiver from the two opposite directions not quite simultaneously, since in practically every case the waves would have travelled different distances. The result is that each individual sound is reproduced twice, with a very slight time-lag amounting, for example, to perhaps one-fourteenth of a second in India. A reflector prevents this and at the same time increases the strength of reception in the main direction by about 35 per cent. in practice, but at the same time reduces the possibility of useful reception in the reverse direction. Echo, however, does not by any means invariably occur, and a certain combination of circumstances is necessary before radiation is propagated in both directions with sufficient strength to cause a noticeable effect.

The question which has occupied the attention of the B.B.C. most in connexion with short-wave aerial design is the determination of the optimum angle with the vertical at which the waves should be projected into space. This angle can be controlled by adopting

a suitable design. Since all short-wave transmission depends on reflection from the ionosphere, and since the efficiency of reflection depends on the angle of arrival of the waves at the ionosphere, the question is obviously extremely important. A great deal of work has been done by other organizations, and naturally this has also been taken into account. It is, however, far more difficult to determine the optimum vertical angle for all practical cases than the horizontal angle. The latter must depend mainly on the number of transmitters for which financial provision can be made, since the narrower the beam, the stronger the reception but the smaller the area served. The method adopted by the B.B.C. for determining vertical angle has been a severely practical one, and has consisted of transmitting with a number of different aerials under conditions allowing a direct comparison of reception at the distant end. It has been necessary to collect a large number of reports and to make many changes, extending over about 2 years. It will be realized that for each direction of transmission it is necessary to provide several aerial arrays in order to follow the different conditions of season, etc., since a separate array is required for each wave-length. Eventually a stage was arrived at when it was possible to work out a complete aerial system covering all transmissions under all conditions of light and darkness, season, and solar cycle. This is, of course, far more elaborate than the simple and inexpensive system with which the service was begun on a tentative basis 4 years ago. At that time the question as to whether such a service would be a success was in some doubt, and for this reason the amount of financial provision justified was naturally limited. At the present time experimental aerials are in use, while a new system of aerials is in course of erection. The total number of separate aerial arrays will be twenty-four. Various devices have been developed to reduce the number of masts necessary to support these arrays, such as an electrical device to make it possible to transmit in slightly different directions from one fixed array. The total number of masts involved is twelve, and the area of land occupied amounts to about 150 acres.

When the service was opened two transmitters were installed, each capable of radiating between 10 and 15 kilowatts; this was the maximum power for which suitable designs existed at that time, and it was not considered desirable to wait for further development. The Marconi Company and Standard Telephones & Cables have since developed designs to a performance specification prepared by the B.B.C., and three additional transmitters—each of which will radiate some 50 kilowatts—are in course of erection. These transmitters are provided with means for a quick change of wave-length, since this process has to be carried out

every few hours. The same means for economizing power is used as with the latest transmitters in use for ordinary broadcasting. The new aerial system and transmitters, which are expected to be in operation early in 1937, should have a marked effect on reception overseas.

Television.

It will not be possible in the short time remaining to consider in detail all recent developments in broadcast television. I shall, therefore, endeavour to indicate the relatively recent advances which have led to the establishment of a service in London. As in the case of sound-broadcasting, it is probably desirable to begin by reviewing very briefly the general problem which has to be solved. The transmission of an image of moving objects, or a cinematograph film, can be looked upon as the development of the transmission of a static picture or design, sometimes called facsimile transmission. Let us assume, therefore, that we wish to transmit first of all an ordinary black and white picture by wireless. If we imagine that a large number of horizontal invisible lines are ruled very close together and from left to right across the picture, it is clear that, beginning at the top and working downwards, each line would pass through areas of varying blackness. It is possible to produce an electric current the strength of which varies with the degree of blackness along each line taken in turn, a process which is known as scanning. Moreover, there is no reason why this current should not be transmitted to a distance in exactly the same way as the varying current from a sound microphone. Again, at the receiving end apparatus could be provided which would draw horizontal lines of a blackness which is controlled by the varying strength of the transmitted current. If an electric pulse is sent out from the transmitting end when the end of each line is reached, causing the reproducing stylo at the receiving end to return, without marking, to the beginning of the line, and if at the same time it is moved downwards automatically by an amount equal to the width of a line, the process can be repeated until all the lines have been traced out one by one and the bottom right-hand corner of the picture is reached. A complete reproduction of the picture would then have been transmitted and reproduced at the receiving end. This is, in fact, a very elementary description of the process of telegraphing a picture.

For television—that is, the reproduction of an image of a moving object or cinematograph film—we must go a stage further. Imagine that another kind of pulse distinguished from those at the end of each line—say of longer duration—is sent out, when the reproducing

stylo has reached the end of the last line, causing it to return, again without marking, to the beginning of the first line. This process could be repeated with any desired frequency, such as 25 or 50 times a second. Further, if the stylo is replaced by a minute spot of light travelling across a screen, such as that caused by a fine electron stream projected on to a screen of fluorescent material, each picture would disappear as quickly as it was drawn were it not for the phenomenon known as the "persistence of vision" in the human eye which causes it to remain imprinted on the brain for a very short period, of the order of $\frac{1}{50}$ second. The result of this phenomenon is that, if about 50 pictures a second are transmitted, the eye sees one apparently almost steady picture. Finally, if instead of to a simple stationary picture at the transmitting end, the process is applied to a moving picture similar to that produced on the screen by a cinematograph film, moving pictures are reproduced at the receiving end. The greater the number of lines into which the picture is divided, the finer will be the definition, and the greater the number of times the whole picture is scanned and transmitted, the steadier will be the picture within certain limits. An increase, however, in either of these quantities involves an increase of the frequency-band width of the transmitted current.

The early attempts at television in Great Britain were mainly on a basis of thirty scanning lines per picture and twelve and a half pictures per second. In these circumstances very little definition was possible, and the low picture-frequency was visible as an irritating flicker. Mechanical methods were used both for scanning the picture and for reproducing it. Low-definition television of this kind can be transmitted on ordinary broadcast wave-lengths of the order of 400 metres but it is not considered to have any value for a broadcast service. The necessary number of lines per picture and pictures per second must be, to a certain extent, a matter of opinion. Moreover the two constants must be balanced against the width of ether channel which is necessary to transmit the rapidly varying current.

At the London Television Station there are two systems, using two hundred and forty lines with twenty-five pictures per second, and two hundred and two and a half lines with fifty pictures per second. These have been developed by the Baird Television Company and the Marconi–E.M.I. Television Company respectively. The latter system employs a method known as interlaced scanning, whereby each scanning of a picture is carried out with two hundred and two and a half lines, and each consecutive scanning is intermeshed. The object of the half line is to give the necessary displacement. Thus, from the point of view of the definition obtainable, the effective number of lines into which the picture is divided is

four hundred and five. The object of interlacing is to obtain high definition together with a high picture-frequency of fifty per second, whilst occupying the minimum possible band-width in relation to performance.

It is now necessary to consider the conditions under which high-definition television can be transmitted by wireless. It has been stated that in the case of low definition an ordinary broadcast wave-length can be used, the reason being that the necessary channel-width is comparable with that necessary for the broadcasting of sound. For the transmission of high-definition television, such as that for which the London Television Station is equipped, the modulation-frequency spectrum goes up to something of the order of 2,000,000 cycles per second, that is to say at least two hundred times that required for sound-broadcasting. This means that the space taken up by a television transmitter is some two hundred times as great as that necessary for sound, and as a consequence there would not be room for one transmission of television over the whole broadcasting band of from 200 to 545 metres. This is a band which could accommodate about one hundred sound-broadcasting stations without mutual interference. In practice it accommodates far more, but not entirely without interference. Again, even if the space were available there would be extreme difficulty in designing apparatus to cover such a band-width on these wave-lengths. Thus it becomes necessary to go to much shorter wave-lengths where the band-width problem is less acute, and at present it is doubtful if wave-lengths much above 7 metres could be used satisfactorily.

Until a year or two ago very little was known as to the behaviour, on a practical basis, of wave-lengths of the order of about 6 metres (the wave-length of the vision-channel at the London Television Station is actually 6·67 metres). It was known, however, that it was difficult to generate and radiate large amounts of power. It was considered also that the range for good reception would be limited by the distance of the horizon, as viewed from the level of the aerial system. At the same time areas of bad reception at distances within this range were anticipated, due to the electrical screening effect of high buildings and hills. Electrical interference, caused by spurious radiation from the ignition systems of motor-cars and from electro-medical apparatus, was known to exist, but the extent of the inconvenience likely to be caused was difficult to estimate. It will be interesting to examine how far these early anticipations have been confirmed or otherwise. Firstly, with regard to the generation of power, the two firms responsible for the design of the transmitters at the London Television Station found it possible to produce vision-transmitters capable of radiating a peak power of

about 17 kilowatts on a service basis. It is now agreed by most engineers interested in this subject that the distance of the horizon is not at any rate a critical factor in determining range. Attenuation is, of course, far more rapid than on a wave-length of 400 metres, but it must be remembered that a far smaller field-strength is necessary for good reception, owing to the absence of interference from natural atmospheric radiations on very short waves, which

Fig. 11 (a).

Field-Strength Survey at 2·5 Metres above Ground.

constitutes to a large extent a compensation. It will be remembered that this applies to a considerable extent to the wave-lengths used for long-distance broadcasting (13 to 50 metres) in relation to those used for ordinary broadcasting. It should be noted also that, in the case of wave-lengths of the order of from 6 to 7 metres, very little reflection at the ionosphere takes place, and it is only under somewhat critical conditions that long-distance indirect-ray reception is observed. In any case there is little practical prospect of long-

distance picture-reception being possible, owing to the distortion which would occur en route.

Figs. 11 (a) and *11 (b)* show the results of a field-strength survey of London and district for the sound-transmitter at the London Television Station, using a carrier-frequency of 41·5 megacycles per second and a power of 3 kilowatts. *Fig. 11 (a)* shows the field measured near ground-level, while *Fig. 11 (b)* shows the estimated field at roof-

Fig. 11 (b).

ESTIMATED FIELD-STRENGTHS AT 10 TO 15 METRES ABOVE GROUND.

level, where the receiving aerial would normally be placed. For the broadcasting of a television programme it is, of course, necessary to use two transmitters, one for vision and a separate one for sound. The latter was used to make this survey because its carrier wave remains constant during transmission, a condition not applying to the vision-transmitter. The field-strength contour lines on the map show that the distribution is irregular in some districts, but not to the extent which was anticipated before the station

was built. It must be pointed out, however, that a series of readings taken along a street lined with fairly high buildings shows enormous variations. On the other hand, if the readings are taken on the roofs of the buildings, they will be found to be much higher in value and more regular. This is important, since aerials are usually erected on the roofs of buildings. The effect of interference from the ignition systems of motor-cars has been found somewhat serious at distances of more than 4 or 5 miles from the transmitting station, becoming, of course, more so as the distance is increased. The effect of a single motor-car is confined to a small area, so that even at distances of 20 or 25 miles from the transmitter the trouble is obtrusive only when a car is opposite a house where reception is being carried out. Conditions on a main road might, however, be bad at any considerable distance from the transmitting station. Reasonably simple means exist for the prevention of these parasitic radiations, and it is to be hoped that means will be found in the future for ensuring that all motor-cars are fitted with suppressors. It should be mentioned here that this difficulty does not occur on ordinary broadcast wave-lengths, so far as motor-cars are concerned, since the spurious emissions do not occur on wave-lengths as high as 200 metres. Similar interference from electro-medical apparatus is more serious, but, of course, much less widespread. Its cure is far more difficult, but time will not permit of consideration of the problem involved.

I shall now attempt to describe very briefly the practical means which are employed to produce the modulation currents—in other words, to convert film pictures or images of living objects into electric currents in the same way as sounds in a broadcast studio are converted into equivalent electric currents by the microphone. It should be said straight away that the apparatus is extremely complicated and nothing more than a mere indication of methods will be possible. Moreover, it is in a state of evolution and development.

It is convenient to begin with that part of the apparatus at the London Television station, at the Alexandra Palace, for which the Marconi–E.M.I. Television Company was responsible. This provides for the transmission of indoor and outdoor scenes and of cinematograph film. Indoor studio scenes require about the same intensity of artificial light as is used in a film studio, whilst outdoor scenes are satisfactory if the light is equivalent to what is required to take an ordinary news-film. The method employed centres round the use of what is called an electron camera, its trade name being the "Emitron" camera. In this device there is a glass vacuum tube, somewhat resembling a cathode-ray tube. One end is widened out and on its interior surface there is a mosaic of light-sensitive

cells, that is to say, cells whose electrical activity varies with the amount of light falling upon them. The tube is so shaped that by means of lenses similar to those used in an ordinary camera, the image of the scene to be televised can be focused on to the inside surface of this screen. The tube also contains a filament and means of focusing a very fine electron beam on to the screen. There are also the necessary control electrodes in the tube, by which the beam can be made to scan the screen and therefore the image which has been cast on it. Apparatus external to the tube is required to generate the necessary continuously-varying voltages for the control electrodes. The instantaneous current flowing through the tube via the filament, the electron beam, and the light-sensitive mosaic screen will be proportional to the amount of light falling on the particular cell on which the electron beam rests at that instant. Thus, as the beam travels from cell to cell, the current varies in proportion to the amount of light falling on each cell, so that, in effect, its variations correspond to the variations of light and darkness along the lines traced horizontally across the image cast on the screen. Moreover, during the time the beam is not falling on any particular cells they are storing up energy corresponding to the light-intensity falling on them. This energy is available for discharge as the beam reaches each one in turn. This fact greatly increases the sensitivity of this type of tube. In this way a pulsating current of very high frequency is produced, involving a maximum frequency of the order of 2,000,000 periods per second. This current passes through a series of amplifier and correction devices of a very complex nature and, eventually, together with the line- and picture-synchronizing pulses, modulates the carrier wave of the transmitter. When a film is being transmitted the process is similar, the film being projected on the screen of the Emitron camera in the same way as the image of an actual scene. Very broadly speaking, the design of the vision-transmitter is on conventional lines, but it contains many ingenious devices to cater for the special requirements of the wide modulation band. It also differs from an ordinary broadcast transmitter due to the fact that the carrier wave is radiated on the very short wave-length of about 6 metres.

The method employed in the Baird process is entirely different; the scanning of films is carried out by a mechanical method which is a highly-developed form of the well-known Nipkow disk. The disk itself rotates at the very high speed of 6,000 revolutions per minute and, in general, the effect is to cause a fine beam of light to scan and penetrate the film. The intensity of the light passing through the film at any instant corresponds to the density of the film picture at the point where it passes through. This varying

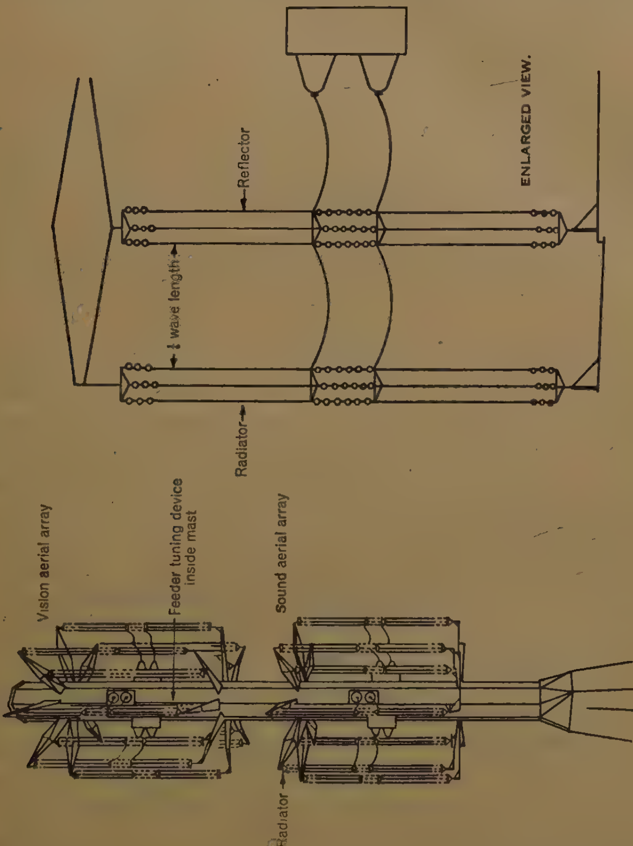
beam of light is then made to fall on a device which generates current corresponding to the intensity of light falling on it. This varying current is then amplified and combined with the line- and picture-synchronizing pulses, and passed to the transmitter modulating circuits. When a studio scene is to be televised the method is somewhat similar, the scene being first of all photographed on to a film strip, which is rapidly developed, washed and fixed, and immediately scanned while still wet by the method just described. Special apparatus has been designed for very rapid development of the film, and the whole process is complete in 1 minute. Thus, during the process of transmitting by this method there is a time lag of about 1 minute between the moment of photographing the scene and transmitting the result through the wireless transmitter. The sound which accompanies the picture is recorded on the side of the film and is subject to the same rapid-development process, and, of course, the same time delay.

There are two other methods used in the Baird system for direct television. The first is known as the spotlight scanner, and its principal use is for transmitting the head and shoulders of a speaker. This is essentially a development of the system used for low-definition television several years ago. Very briefly the process is as follows. An intense spot of light is made to scan the subject directly by means of rotating-disk apparatus of the Nipkow type. The light reflected from the subject falls on batteries of photo-electric cells, setting up a constantly varying current corresponding to the amount of light reflected from the subject as the spot of light carries out the scanning process. Naturally it is necessary for the studio to be practically in darkness in order that the only appreciable light falling on the photo cells shall be that due to the reflection of the travelling spot of light. The second method involves the use of an electron camera, analogous to but differing very widely from that mentioned in connexion with the Marconi–E.M.I. system. The electron tube in this case is as yet in the research stage, and has only been used experimentally at the Alexandra Palace station. For this reason it is not proposed to attempt to describe its working. At the time of writing it is being used for studio scenes as an alternative to the intermediate film method.

The Baird vision-transmitter differs considerably from that used in the other system already described, and incorporates an unusual type of thermionic valve in which no glass is used. This valve is made entirely of metal, is demountable and is continuously evacuated by pumps capable of maintaining the very high degree of vacuum necessary in all thermionic valves.

A common aerial is used for both systems, consisting of a series

of vertical dipoles arranged in the form of a cylinder around the top of a steel mast. There is an inner circle of reflector wires, each dipole having its own reflector element. In this way the steel structure of the mast is shielded from the intense electric field, and therefore cannot interfere seriously with the radiating characteristics of the aerial system as a whole. The mast itself is an unstayed structure



Figs. 12.

VISION AND SOUND AERIALS AT ALEXANDRA PALACE.

220 feet high, mounted on and anchored to one of the four existing brick towers at the Alexandra Palace. The apex of the mast is approximately 300 feet above ground-level, and 600 feet above sea-level. *Figs. 12* show an outline of the arrangement of mast and aerials. The enlarged view shows one dipole radiator element and its associated reflector. There are eight elements in each aerial. It is especially important in the case of ultra-short-wave transmission that the aerial should be raised as far as possible above

ground; in fact, the strength of reception varies with the height of mast. The site chosen should also be as high as possible above sea-level; in fact, the ideal position for it would be the highest point in the area which it is desired to serve.

A third transmitter is provided to radiate the sound which accompanies the vision programme. This is, of course, equivalent to the sound associated with an ordinary cinematograph film. This sound-transmitter is used with either vision system in conjunction with an aerial similar to that used for vision and mounted on the same mast 30 feet below.

Thus, the complete equipment at the London Television Station consists of two separate television systems, each complete in itself, with the exception of the aerial and the sound-transmitter, which are used in turn with each vision equipment. The object of this plan is to enable the two systems to be operated under conditions suitable for a direct comparison under practical conditions of service.

The question of receivers for sound and television broadcasting falls outside the scope of this Paper, but in the case of television it is perhaps desirable to mention briefly the lines on which the majority of receivers at present in use have been developed. It will be remembered that when an elementary description of the process of television was given earlier in this lecture, it was mentioned that the picture could be reproduced at the receiving end by a spot of light travelling over a screen, the intensity of which varied continuously with the light and darkness along imaginary lines across the picture, and that the spot of light might be made by a fine stream of electrons falling on a fluorescent screen. This is the principle employed in an ordinary cathode-ray tube of the kind which in these days is used very extensively for scientific measurements. Such a tube forms the main feature of the majority of present-day television receivers. The tube itself is usually of the normal shape, similar to a flask with a nearly flat end, and on its inside surface is the fluorescent material. The filament and control electrodes are assembled at the opposite end of the tube, and the fine pencil of electrons, which is focused on to the screen, can be made to scan it with the correct line- and picture-frequency. The line- and picture-frequencies are kept synchronized with the transmitter by means of pulses sent out from the latter, as already explained. The tubes vary in size up to a diameter at the screen end of about 15 inches. It is, unfortunately, not possible to make the screen exactly flat, since it must be in some measure convex in order to resist the danger of implosion and to assist in maintaining uniformity of focus over the whole area of the screen. The receiver includes a

unit for reproducing the sound, and it is usual for the tuning to be coupled in such a way that it is only necessary to operate one knob to tune in both the sound and vision transmissions.

It is hoped that this Paper will enable engineers engaged in other branches of the profession to form some picture of the engineering activities of a modern broadcasting organization. It has only been possible to describe the bare outline of the problems and processes involved, and obviously it has been necessary to omit all non-essential details. For those for whom this Paper is intended this may not be a disadvantage, since even after 14 years the technique, so far as details are concerned in most branches of the work, is constantly and rapidly changing. In conclusion I should like to express my acknowledgment to the Director-General of the B.B.C.—himself a member of The Institution—for permission to read this Paper.

The Paper is accompanied by fifteen tracings, from which the Figures in the text have been prepared.

Discussion.

The Author. The AUTHOR showed a series of lantern-slides illustrating his Paper.

The President. The PRESIDENT, in moving a vote of thanks to the Author, said that the Author had given them an extraordinarily interesting Paper. The progress of broadcasting and television during the last few years had been very great, and a tremendous amount of work on those subjects had been done by the B.B.C.

Mr. Donkin. Mr. S. B. DONKIN said that, before discussing the Paper, he would like to explain that, at the request of the Council, it had been prepared purposely to be read before an audience composed of engineers who were not wireless experts. He was sure that the general survey of present practice had been appreciated, and he congratulated the Author on the way in which he had dealt with the subject.

In the earlier part of the Paper reference was made to the congestion of the available broadcast-channels and of the steps which were being taken by the B.B.C. to improve matters, including the employment of synchronized transmitters for some National programmes; the Author had admitted that that was a sound arrangement in theory but that it only gave "an almost positive locking" of the two frequencies in practice. Those synchronized transmitting stations appeared to be coupled by means of telephone-cable, and the synchronism between them was clearly as good as appeared possible. Could the Author state whether the defects in reception at the fringe of two broadcasting areas were now eliminated, and, if not, whether transmission on ultra-short waves might be introduced in the near future?

The description given in the Paper of the new ribbon-type microphone was of great interest. Was the sensitivity of the ribbon microphone great enough to enable it to be placed in the position in relation to an orchestra normally occupied by the audience? If that were possible the results ought to be as good as they would

Mr. Donkin.

normally be in a concert hall, provided that the studio was correctly proportioned and treated in regard to its wall-surfaces. Did the use of that microphone still render necessary the manual control of the balance of sound from various instruments and the control of the range of amplitude of orchestral sound, or was the transmitter still insufficiently developed to render that necessary? To give an example of the difficulty, when listening to an orchestral broadcast from the B.B.C. London studio normal reception was apparently obtained from the full orchestra and it was also possible to hear, quite properly and adequately, the solo parts; in the case of a relayed programme recently, however, when the receiver was controlled to give the normal reception of the full orchestra, the solo parts could hardly be heard at all.

The announcement by the B.B.C. so soon after the establishment of their television service that they had decided to standardize television broadcasting on the Marconi—E.M.I. system would no doubt be of considerable advantage to manufacturers and to the public. The part of the Paper dealing with television appeared to be particularly interesting in so far as it enabled those not well versed in the subject to understand to some extent the reasons which had made the B.B.C. decide to use the Marconi—E.M.I. system.

Sir George Lee.

Sir George Lee remarked that the Post Office system acted as an auxiliary to broadcasting in several respects. An important function which the Post Office performed for the broadcasting organization was that of reducing the interference which many listeners experienced on their wireless sets from electrical machinery of various kinds, such as refrigerators, motors of different sorts, and electrical medical apparatus; while, as was indicated by the Author, television was now being interfered with by the sparking-plug system on motor-cars. Last year the Post Office had had about 40,000 cases of interference to investigate; they had not succeeded in curing all of them because at present they had no legislative power to take action, but they had succeeded very well in most of the 40,000 cases which were actually found to be causing trouble, by persuading the owners of the apparatus concerned to introduce some means of suppressing the interference. There had been a Committee of the Institution of Electrical Engineers sitting for some time on the question of interference, and they had recommended that legislative powers should be granted to the Electricity Commissioners to draw up regulations on the subject; it was intended that, if the Government approved such a course, the Electricity Commissioners would have the duty imposed upon them of drawing up the regulations and the Post Office should undertake the duty

Sir George Lee. of enforcing the regulations. That stage had, however, not yet been reached.

The Author had mentioned the noise-factors in the circuits used for broadcasting. All electrical-communication circuits picked up noise, or cross-talk, from each other. The Post Office reduced that as much as possible. In the case of broadcasting they had to provide circuits specially screened with tin-foil or some such material to prevent the circuits interacting on one another; by that means amplification could be increased and the circuits could be made very much quieter. The total circuit-mileage used by the B.B.C. in their net-work last year had been about 6,000, and in addition they had asked for special circuits to be arranged for 4,600 special events. A large number of circuits were also in use for continental broadcasts, either from Great Britain to the continent or *vice versa*. Every country in Europe had those special circuits, which were often used for the broadcasting of special events. Many programmes were transmitted to and from America over the trans-Atlantic telephone circuits.

One new development (although it was not yet in use) had been the laying of a new type of cable between London and Birmingham, known as the "coaxal" type of cable. It consisted essentially of a copper tube about ½ inch in diameter containing a central wire. The insulation was very largely air, and there were four tubes in the one cable. Two of them were intended to transmit television broadcasts. The cable was being extended to Manchester in the current year's programme, and it would eventually be continued to the northern towns. There was as yet no television station in Birmingham or in Manchester, but when it was erected the Post Office would have their cable ready to transmit television to those places. The cable was especially interesting from a telephone standpoint, but that was not a subject within the range of the present Paper. It was hoped to be able to have 300 telephone talks proceeding on those tubes simultaneously, and for television it was hoped that the cable would carry a frequency of 2,000,000 cycles per second, which was apparently what was required at the present time for giving good television.

The Author had referred to the power used in broadcasting, and one of Sir George's assistants had calculated the amount used for reception. It was estimated that, if 80 per cent. of the mains operated receivers were in use during some important event such as the Coronation (when most people switched on their receivers) the load on the grid system would be 250,000 kilowatts due to receivers alone. The annual consumption of receivers was estimated by his

staff to be 270,000,000 units. Although it was not a big percentage, it formed quite a useful addition to the load on the grid. Sir George Lee.

Another interesting statistical fact in connexion with the broadcasts to and from America was as follows: there were three short-wave and one long-wave stations working to the United States every day, and altogether several hundred kilowatts were radiated from Great Britain towards America, but all that was picked up in America was sufficient energy to raise a fly 7 inches in 1 year! On land-lines amplifiers were put in at about every 50 miles, and there was, therefore, a large number of amplifying stations between Great Britain and, say, San Francisco. Each amplifier received such feeble signals that they were just distinguishable from the inherent noise on the circuit, and it amplified them sufficiently for the signals to be received at the next station. The total amplification entailed in that repeated process was 10^{256}, and he believed that that figure was too large to describe any single quantity in the Universe.

Mr. C. E. STRONG remarked that the section of the Paper dealing with studio design and technique was bound to cause many to glance askance at the acoustic conditions under which they received the programmes. It was not usual to listen in rooms which had been scientifically treated from an acoustic point of view, and it would be of interest to know to what extent the praiseworthy efforts of the B.B.C. were rendered nul and void on that account. He had seen a room in a private house in which all the precautions suggested by modern studio-technique had been applied, but he did not know whether the results gave satisfaction. Was it reasonable to expect any degree of satisfaction to justify such an outlay? Mr. Strong.

The aim of the broadcasting engineer was presumably to create the most perfect illusion of reality, but the sense of the width and depth of the orchestra was lost through the comparative absence of aural perspective when sounds were collected and transmitted to listeners over a single channel. It would be possible to restore aural perspective to a considerable extent by transmitting simultaneously from two spaced microphones, over separate channels to a pair of suitably-spaced loudspeakers; that had been done successfully in transmitting from one large hall to another, but it was questionable whether such a scheme was practicable under the usual restricted conditions of broadcasting reception. Up to the present it had been hardly worth considering on account of the practical difficulty of providing two channels for one transmission, but with the introduction of ultra-short-wave transmission as used for television and its accompanying sound programme it appeared that it might be worth while to consider whether such a scheme could be introduced.

Mr. Strong.

Further, the illusion of reality was limited to some extent at prese by the necessity of contracting the range over which the loudness the programme might vary, in order to pass the programme throug the bottle-neck of the broadcasting transmitter. The programm was flatter at the output of the loudspeaker than an orchestr conductor would choose to have it. The contraction was effecte at the transmitting end by manual control, but if it were dor automatically then it would apparently be feasible to incorpora in receivers complementary automatic expanding apparatus whic would restore the whole volume-range of the original programme.

The band of musical frequencies that could be received was som times undesirably limited owing to the overcrowding of the ether the waves used travelled, in fact, too far and made themselv apparent where they were not wanted. Ultra-short waves as use at the Alexandra Palace did not travel so far, however, and it wa interesting to speculate if they were likely eventually to be used f the Regional services. The National service could still be carried c by one or two stations of the Droitwich type. If all countries we to use ultra-short waves it would not be possible to receive so mar foreign programmes, but there would be less interference of or programme by another.

The Author had mentioned that some controversy existed as the best form of vertical radiator for obtaining the maximu reduction of high-angle radiation. One of the questions was t importance or otherwise of uniform cross-section of the mast.

Radiators for the longer wave-lengths in the medium wave-ban say for a wave-length of 500 metres, had to have a height of fro 800 to 1,000 feet. For such heights uniform cross-section practical implied guying at a number of points up the mast, and it wa quite possible that the advantage of uniform cross-section, if an might be lost by reason of electrical reactions from the guy-cable even though their lengths were broken by insulators. The radiat of the Budapest station, for example, was of tapering section ar guyed only at one point. It worked on a wave-length of 5 metres and its height was 1,030 feet, which exceeded that of t Eiffel Tower. The section was square and the length of a side the largest section was 48 feet. The guys, of which there were eigh were of specially pre-stressed steel cable having a diameter $2\frac{1}{4}$ inches. The static stress on each guy was 32 tons, rising 150 tons in service. The weight of steelwork was 230 tons and t total static load on the base insulator was 480 tons. He understo that the satisfactory transmission results obtained with that radiat would tend to support the view that the variable cross-section w

not of much consequence, but the difficulties of appraising exactly the anti-fading properties of an antenna were such that the final answer might not be found for some time. Mr. Strong.

Mr. M. T. Tudsbery, referring to the masts, said that the B.B.C. had some 10,000 feet of stayed masts in service as compared with 1,000 feet of self-supporting masts. Would the Author give reasons why the B.B.C. preferred the stayed mast to the self-supporting mast, seeing that in America the self-supporting mast was used to a considerable extent? The wind-loading of the Lisnagarvey mast (*Figs. 7*, p. 363) had been calculated on the basis of 27 lbs. per square foot at its base, rising to 47 lbs. per square foot at its top. There was a single winch at 115 feet elevation which raised and lowered the topmast, which was 75 feet long. The topmast was tapered, the maximum and minimum sections being 16½ inches and 10½ inches in diameter respectively, and there had been some difficulty in determining a suitable method for holding it. Ultimately it had been found possible to adapt an ordinary self-centering lathe-chuck to hold the topmast in position as it was lowered or raised. On p. 364 the Author compared the ordinary parallel-sided mast, such as was shown in *Figs. 8*, with the Lisnagarvey mast, and stated that the parallel-sided mast ". . . appears to be at least as effective, although it may possibly prove to be electrically less robust." Further details on that point would be of interest, especially in regard to the statement which the Author made later that "A somewhat similar mast, 475 feet high, is under construction . . . at Stagshaw." The Stagshaw mast was going to have only three stay-points in its height. Extraordinary evidence of long-column action had been experienced by the B.B.C. One of their masts erected on the Yorkshire moors had been exposed to an exceptionally severe gale, which had found a weakness in one of the bottle-screws which connected the stays with the anchorage-blocks. The bottle-screw had failed, with the result that all the insulators in the stays to windward of the mast had failed in sympathy, except in the lowest stay and in the uppermost stay. Each insulator which had failed had naturally allowed the loops around the insulator to draw together, with the result that the mast had bowed out to an extent of about 4 feet 6 inches. It had not failed, however, and it was brought back into its proper alignment when the gale had subsided. He thought that the fact that a 500-foot column of triangular-section lattice-steel could withstand such punishment was very surprising. Mr. Tudsbery.

In conclusion, he asked the Author to give some information of the conditions which ruled and governed the transmission of broad-

Mr. Tudsbery. casting waves through or around, above or below, those places on the earth's surface which did not seem to be amenable to the reception of wireless signals within a service area.

Dr. Walmsley. Dr. THOMAS WALMSLEY pointed out that, in the curves of reverberation-time given in *Fig. 3* (p. 354), the ideal curve stopped short at 8,000 cycles per second, whilst on p. 360 it was stated that the frequency band should extend to between 6,000 and 9,000 cycles per second for broadcasting purposes. Again, on p. 366 it was stated that good-quality broadcasting required a frequency of 7,000 cycles per second, and that the highest reproduction needed 10,000 cycles per second, or more. He thought that Mr. Donkin had been perturbed by the question of the frequency-band required, and he asked the Author whether he considered that a discriminating listener, who had not got comparative means of testing, was able to distinguish between a programme received at 7,000 cycles and one at 10,000 cycles per second. If there were two loudspeakers, one having a broader band-width than the other, and direct comparisons were made, it was then quite easy to detect the difference between certain classes of music; such sounds as the clanging of cymbals, for example, or orchestral music, or any types of sounds which were rich in transients, showed up much better with loudspeakers capable of reproducing high frequencies. On p. 360 the Author stated that the tolerable level of spurious noise for broadcasting was about one-twentieth of that experienced with commercial speech. Was that an energy ratio or a voltage ratio?

Particulars were given on p. 362 of the object of the ring at the top of the Lisnagarvey mast. Some years ago, when the Rugby radio-masts were being designed, he had had occasion to make careful calculations into the effect of raising the mast above ground; the calculations showed that beyond a short distance above the ground the capacity due to the vast mass of metal had a preponderating effect and that it would have needed a ring of very large dimensions at the top of the mast to make much difference in the total capacity. If the top of the mast were raised a considerable distance the electrical wave-length of the mast would be altered. Had any tests been made, however, to ascertain whether reasonable increases in the diameter of the ring made much difference in the electrical characteristics of the masts? From his own calculations he was of the opinion that they would not.

On p. 371 reference was made to echoes. Echoes were commonly experienced at certain times of the year, and were due to the fact that some radiation passed right around the earth and then reached the receiving station, so that the receiving aerial obtained energy

from both directions. Perhaps the Author would state when he experienced the greatest amount of echo-effect. The Post Office had found, from tests extending over several years, that echo-effect occurred most markedly about November and continued during the winter, but that towards the spring it seemed to disappear. Dr. Walmsley.

The question of wave-length allocation was receiving considerable attention at the present time. There had been a suggestion that wave-length allocation should extend to wave-lengths which were shorter than 10 metres. Up to a year or two ago it had been thought that waves with a length below 10 metres could only be received close to the transmitter. Recently, however, it had been stated that the sound-emissions from the B.B.C. television programme had been actually received in South Africa. It was known, too, that certain police transmissions in the United States had been received strongly at Dollis Hill. Could the Author state what was the experience in America on the ultra-short waves transmitted from Great Britain? Were they receiving them? If they were, were they receiving them in good strength, and were they receiving them sporadically or at all times of the day during certain periods? The matter was extremely important, because if those ultra-short waves had long-reaching properties, naturally they were bound to be allocated on an international basis.

***Mr. R. M. Chamney pointed out that ordinary telegraph and telephone communications did not normally require frequencies below about 250 periods per second, and amplifiers could therefore be constructed on a simple principle using the small type of transformer normally associated with telephone engineering. Music transmission had, however, completely altered the picture from the telephone engineer's point of view. Bass instruments, such as drums, required frequencies as low as 50 periods per second, and it was impossible to pass them through the normal telephone amplifier without great loss of volume at those frequencies. At the opposite end of the range, whereas telephony did not require anything above 3,000 periods per second, instruments such as violins required at least 7,000 periods per second to prevent an unnatural tone from being produced. He had been present at some interesting experiments in Berlin in the early days of international broadcasting; a violin had been played and a microphone picking up the sound had been connected to a loudspeaker in another room. It had been possible, by means of wave-filters, to alter the frequency-band Mr. Chamney.

***This and the succeeding contributions were received in writing. —Sec. Inst. C.E.

Mr. Chamney. received in the loudspeaker. When first the violin had been heard all frequencies above 5,000 periods per second had been cut off, and the audience, knowing that a violin was being played, had been quite satisfied. The frequency-band had then been extended to about 8,000 periods per second, and immediately it had been realized that that reproduction was much more faithful. The frequency-band had then been again restricted to 5,000 periods per second, and the violin had become, to all intents and purposes, a flute. That experiment emphasized the need for a wide frequency-band in order to obtain good reproduction of music.

It was therefore essential for amplifiers fitted in telephone-lines used in broadcasting to be capable of transmitting frequencies between 50 and at least 7,000 periods per second without distortion of amplitude. The British Post Office had solved that problem by using a special type of amplifier so designed that both the lower and the higher frequencies were dealt with faithfully. That was chiefly due to coupling the stages of amplification by resistance and capacity instead of by the normal transformer arrangement. The output valves of the amplifier were arranged as a push-pull connexion in order to obviate the non-linear distortion, it being impossible to produce valves which had a straight characteristic curve.

A further difficulty from the telephone engineer's point of view was that the broadcasting of music imposed a very much greater strain on the plant owing to the range of volume being very much larger than was met with in speech. That had already been referred to by Mr. Donkin. It was necessary to arrange that amplifiers had a sufficient range of amplification, so that overloading during peaks of volume would not give rise to the production of harmonics due to non-linear distortion. Sir George Lee had referred to the difficulty which arose due to noise being picked up by the conductors used for the transmission of music and being amplified so as to become audible to the listener. That aspect was also an ever-present trouble to the lines engineer, more especially now that the power engineer was turning to long-distance transmission of high voltages which gave rise to parasitic currents in neighbouring telephone-lines and cables. It was for that reason that the policy of the British Post Office in using conductors screened by means of tin-foil had produced the basically quiet system required by the B.B.C.

Mr .Craddock. Mr. D. A. Craddock observed that, whilst the Paper gave much valuable information regarding the methods used by the B.B.C. to attain the highest practicable standard of quality in its transmissions, it would be of great interest if an overall response-curve could be given representing the highest standard of transmission

at present attained under the most favourable conditions. Such information would be most useful in enabling designers of receivers to decide how far the high-note response could usefully be extended for high-quality reception of local stations; it would be merely detrimental to extend the response-range beyond that of the transmission, as the noise-level in the output would be raised without any advantage being gained. Mr. Craddock.

It would be of great assistance to those interested in high-quality reproduction of broadcasting if the B.B.C. would occasionally transmit a test signal such as a constant-level "gliding" tone or "gliding and warbling" tone covering the whole range of modulation-frequencies transmitted. Valuable measurements could then be made on receivers by means of a simple valve voltmeter or a rectifier-type output-meter.

Mr. Frederick Grisley observed that it appeared that the most desirable development in sound-transmission was to improve the quality of reception by the listener. In view of the Author's remarks as to side-band interference at audio-frequencies above 4,500, did the B.B.C. attenuate frequencies above that in transmission, or if not, what upper limit of level output was maintained? Receiving apparatus was readily obtainable which gave substantially level response up to 10,000 and even 20,000 cycles per second, and it would be of interest to know at what point the present transmissions would fail to do justice to such apparatus, assuming it to be used near to a transmitter where reception free from interference was possible. Mr. Grisley.

The Author stated on p. 365 that no appreciable improvement in the quality of the transmitter-unit had taken place in recent years, mainly because greater amounts of distortion occurred in other units of the system. To what units was he referring? Post Office lines were stated in the Paper to be almost faultless, and the Author mentioned before the Discussion that the difference in quality between input to and output from a transmitter was negligible. Amplifiers and receivers could be made to a similar order of perfection. The remaining units were the microphone and the loudspeaker, and perhaps the high-frequency and detector-circuits of the receiver. Small amounts of distortion in the various units of the chain between the microphone and the loudspeaker might be cumulative, and without relative figures of the magnitude of the loss of quality in each unit, it did not appear possible from a listener's point of view to become complacent with any unit short of perfection, so long as improvement was economically worth while, particularly with those units which were under the B.B.C.'s own control. On certain

Mr. Grisley. occasions the transmission seemed better than usual, and listene might therefore draw the conclusion that, by comparison, the ordi nary standards of transmission were capable of some improvement

Attention had been drawn to the great possibility of ultra-shor waves of the order from 5 to 7 metres in length, which, with th wide spectrum available and the optical limits of service-area, seemed to be ideally suited to a high-quality local service free of interferenc from more distant stations on adjacent channels. Statements had been made that the quality of signals from the television sound transmitter at the Alexandra Palace on a wave-length of 7 metre was much superior to that of the medium-wave stations, and h would like to know what audio-frequency band was put out from that station as compared with medium-frequency emissions. Wa it to be expected that long-distance adjacent-channel interferenc would occur in the case of wave-lengths below 10 metres? If not there appeared to be no point in allocating such channels as wa done in the case of longer wave-lengths. Was it correct that, i recognition of the high-quality potentialities of the ultra-short waves the B.B.C. had erected an experimental transmitter at Portlanc Place ready to cover London with a high-quality sound-servic when they were satisfied that sufficient apparatus was in the hand of the public to make use of such a service? If that were th case, what were the plans of the B.B.C. in regard to that nev development? Receiving-apparatus for short waves was com paratively simple, and an educational high-quality service would b of great value to the public, whilst a parallel service of that kin would facilitate the gradual change-over of existing receivers to th new conditions.

Could the Author give costs for the erection and maintenance o a modern broadcasting station, as distinct from studio and other en gineering costs? Such figures might assist in giving perspective t the magnitude of the undertaking as compared with other engineerin projects.

The Author. The AUTHOR, in reply, remarked that Mr. Donkin had raised th question of the positive locking of the carrier-frequency of tw stations working on the same frequency-channel. The position wa that, even if the two carrier-waves were locked perfectly, a smal area of poor reception between the two stations could still not b avoided. That was fairly easy to demonstrate on theoretica grounds, but in practice it was not possible to obtain quite perfec synchronization. The carrier-wave of each station would have th same frequency but the waves would not always be exactly in phase There was a slight flexibility which caused a continuous phase swing

'he question had been raised as to whether it would not in the nd be necessary to resort to ultra-short waves. That was a matter which the B.B.C. had been considering very carefully, because it was apparently an obvious solution to certain wave-length difficulties. He would not, however, like to prophesy whether or not such wave-engths would be used in the future, because there was already keen ompetition to obtain the right to use them before their properties ad really been thoroughly investigated. The Author.

Dr. Walmsley had asked whether any reception from the Alexandra Palace was possible in America. It was rather remarkable for that question to have been asked that night, because only that afternoon e had heard a record of a sound-transmission from the Alexandra Palace which had been sent to him by the Chief Engineer of the National Broadcasting Company of America, and reception had een almost perfect. As far as he could make out at the moment, eception "stayed in" for somewhere about 1½ hour at about 0 a.m. in New York.

With regard to the question of the placing of the ribbon microphone, nd whether it was not difficult to find the right position in the studio ecause of its low sensitivity, that was not the case in practice because he ratio between the amount of music picked up by the microphone n relation to the amount of spurious noise had to be considered. If he amount of spurious noise was low it did not matter if the sensitivity f the microphone was also low. In the Queen's Hall, for example, ibbon microphones were suspended at a considerable distance from he orchestra—much farther away than the old microphones had een placed. Mr. Donkin had mentioned the question of depth of ound, or stereophonic effect, and matters which really arose from he fact that a microphone was equivalent to one ear, whereas rdinary people listened with two ears. That was a very abstruse oint, and he could not discuss it in the time available. As to the uestion of limitation of frequency-band in transmitters, the dis-ortion due to any good modern broadcasting transmitter was, ractically speaking, almost nil. A person had to be extraordinarily bservant to tell the difference between the input to the transmitter nd the music as heard after it came out of the transmitter. The mitation of the upper frequencies was actually imposed in other arts of the chain.

The question of the acoustics of the room in which the receiver was ituated had been mentioned. That appeared to be a very important atter, but it was not really quite so important, because although the everberation time of a concert-hall type of studio was about 2 econds, and that of an ordinary fairly small sitting-room was

The Author. perhaps 0·2 second, people usually filled up their rooms with sofa and chairs, and that made the reverberation time in an ordinar sitting-room very low; there was, however, difficulty in the larger typ of house, where there were polished-wood floors and comparativel few soft objects which would cause the room reverberation time t be low. In general, so long as the reverberation time of the room wa very low compared with that of the studio, the music would be hear with the characteristics of the studio without the acoustic propertie of the listening-room having much effect.

The question of interference had recently become important fro the television point of view. It was mentioned in the Paper tha practically all motor cars were capable of causing severe interferenc and at the present time there was no means of ensuring that step were taken to suppress it. If, in due course, the Electricity Com missioners were charged with the work of drawing up regulations i relation to interference with broadcasting generally, no doubt the would consider that very important aspect of the problem. Th other main source of interference to television, namely, that cause by electro-medical apparatus, was much less easy to deal with an that problem might have to be tackled on the lines of allottin definite radio frequencies for medical work.

In connexion with the special coaxal cables suitable for conveyin television signals, it had to be borne in mind that not only would i be necessary to provide such cables to link distant stations wit studios, but also to cover the shorter distances between a trans mitting station and programme-sources outside the studios.

The question of high-quality broadcasting on ultra-short wave and in particular stereophonic broadcasting, had been raise That possibility had been under consideration by broadcastin engineers for several years past. When the subject of stereophoni broadcasting in particular was studied, the question of ether-spac cost, and the benefit likely to be derived by the listener, tended t put the proposition in a somewhat unfavourable light. It had t be remembered also that it would be very difficult to operate system which was partly stereophonic and partly normal. Th development of studio technique to cater simultaneously for bot conditions of reception might even be impracticable. The questio of ether-space was known to be difficult, even so far as ultr short waves were concerned, whilst the cost of operating doub transmitters and duplicate studio gear also constituted a seriou disadvantage. On the receiving side it was rather doubtful wheth a large number of listeners would be prepared to pay the extr cost of a double-channel type of receiver.

The Author.

The question of automatic control of modulation depth was one which had engaged the attention of B.B.C. engineers for some time past. It was, however, almost inconceivable that a system could be devised which would not introduce any distortion at all, at any rate on theoretical grounds. It might, however, prove in the end that the amount of distortion was sufficiently small to make it justifiable to use such a system in order to obtain the advantages mentioned by Mr. Strong. Experimental systems had already been tried, but were not so far used on the home service. On the other hand, for the Empire service such a method had been used for some time past, because in that particular case it was most important to ensure the highest possible level of signal at the receiving end.

The question of cost was bound to enter largely into the consideration of the relative advantages of masts of uniform cross-section, stayed at intervals over the whole length, as compared with those of non-uniform cross-section, stayed only at or near the centre. It was probably correct to say that neither the electrical disadvantages of the existence of a fairly large number of stays, nor the electrical advantage of uniform cross-section, greatly affected the practical performance, and he agreed with the point made by Mr. Strong that it was difficult to appraise the exact value of the various types of mast designed with the object of reducing the effect of fading. Experience so far had shown that self-supporting masts were more costly so far as Great Britain was concerned, and it was questionable whether the difference in performance was appreciable. In America both types seemed to be widely used. The point had been raised as to why the parallel-sided mast should be electrically less robust than that stayed only in the centre. The reason for that arose from the fact that the high-frequency voltage distribution along the mast was not uniform, the voltage between the mast and earth varying progressively along its length. The maximum potential occurred at the top of the mast and the minimum somewhere near the centre, depending upon the exact design and the wave-length in use. If the parallel-sided mast were used, it was essential to employ a set of stays at or near the top, which had to be insulated from the mast to withstand the peak working-voltage, whereas if the stays were attached only near the centre the working voltage of the insulators would be considerably lower.

Mr. Tudsbery apparently referred in his last paragraph to the difficulty of reception in certain areas, sometimes referred to as "blind spots." Such a state of affairs might arise from several local causes, but in general it had to be borne in mind that the ease of passage of a wireless wave over the surface of the earth depended

The Author.

on the electrical resistance of the sub-soil for some feet below tl surface, owing to the fact that the passage of a wave over the surfac of the earth caused currents to be set up in it. Some strata were higher resistance than others; in general, the geologically old formations offered the most resistance. A good example wa afforded in Great Britain by Dartmoor, which offered great resistance to the passage of wireless waves than would normally accounted for by the topographical conditions.

No direct answer could be given to the question raised by D Walmsley as to whether a listener could tell the difference betwe broadcast reproduction limited at 7,000 periods per second comparison with that limited at 10,000 periods per second, unle there was a comparison reference at hand. Clearly that was bour to depend very much on individuals. Many people were unable hear a frequency above about 8,000 periods per second, where others could readily hear up to 15,000 periods per second, or ev higher. Further, it depended very much on the nature of the pr gramme whether the high frequencies were of importance or nc For example, a programme involving what were usually call "effects," such as the noise of machinery, etc., sounded very mu more natural if the upper frequencies were present. Again, a fu symphony orchestra required the reproduction of the upper fr quencies for true musical value. In the case of speech, howev there was no great advantage in reproducing anything above, s 7,000 periods per second, and the same would apply to light music programmes. The figure given for the radiation of spurious noi allowable on a telephone-line used for broadcasting, as compar with one used for commercial speech, referred to voltage.

With regard to the capacity of a ring at the top of a steel mast, t Author agreed with Dr. Walmsley that that would be negligible as addition to the electrostatic capacity of the mast with reference the earth. The mast was a radiator which was, in effect, a grad line having a lumped capacity at the lower end and another lump capacity at the top, and it was the latter which depended on t diameter of the ring. For practical purposes, that varied almo directly with the diameter.

With regard to echo on short waves, the experience of the B.B showed, as would be expected, that that depended on the transmissi path and on the wave-length employed. For instance, it had be experienced on transmissions from Daventry to the east at mid-d through the winter, and was probably worst near the equinox On the other hand, on transmissions from Daventry to the west, had been observed in the early evening in summer. During t

ast winter it had also been reported from Africa on 19 metres during The Author.
ne early evening, and in the morning at Daventry there had been vidence of signals on 14 and 17 metres arriving in Australia over oth paths (long and short) simultaneously. The echo in the latter ase was naturally not so serious, since the path difference was small. Vith regard to the reception of the Alexandra Palace sound transnitter in America, information had been received since the reading f the Paper indicating that the peak field was estimated at about 0 micro-volts per metre.

He would refer Mr. Craddock to curves in two recent Papers.[1] Ie thought that Mr. Grisley's first question would also be answered y reference to those curves.

In reply to Mr. Grisley's point concerning the components in the roadcasting chain in which distortion occurred, it might be said nat the line was usually the point at which limitation of the upper equencies occurred. That was not in any way a reflection on the uality of the lines provided, but arose from the fact that it was not ormally economic to provide long-distance lines which were qualized up to, say, 10,000 periods per second. In the case of local ansmission, where no long line was involved, the usual condition as that frequencies up to about 9,000 or 10,000 periods per second ere radiated at sensibly normal strength from the medium-wave ations. When the receiver was considered it was clearly impossible generalize, beyond saying that the loudspeaker was usually the fender.

With regard to the quality radiated by the Alexandra Palace ound-transmitter in relation to the transmitters at Brookman's ark, it was doubtful whether there was any great difference in the vo cases; at the present time careful tests were being made to tablish that point. It certainly did appear, however, when a mparison was made between the two, using commercial receivers, at the Alexandra Palace contained a considerably greater range the higher frequencies. That, however, was possibly due to the ct that most receivers were designed to cut off at a lower value in e case of the medium waves, in order to exclude adjacent-channel terference.

It was not quite clear what was meant by the question which had en asked as to whether, on long distance ultra-short-wave trans-

[1] *Fig. 17* in "A Wireless Broadcast Transmitting Station for Dual Programme Service," by P. P. Eckersley and N. Ashbridge. Journal Inst. E.E., vol. 68 (1930), p. 1149.
Fig. 38 in "The Droitwich Broadcasting Station," by Sir Noel Ashbridge, H. Bishop and B. N. MacLarty. *Ibid.*, vol. 77 (1935), p. 437.

The Author. missions, adjacent-channel interference would occur. It was hard practicable to reply to that question without a specification of tl conditions referred to, that was to say, of the channel-width allowe and the question of whether possible indirect ray effects were to l taken into account. It was not practicable at the moment to sta whether ultra-short waves would, or would not, be used for tl broadcasting of sound by the B.B.C. In reply to Mr. Grisley's la question, it was not the present practice of the B.B.C. to publis detailed costs for operating the service.

JOINT MEETING WITH BRITISH SECTION, SOCIÉTÉ DES INGÉNIEURS CIVILS DE FRANCE AND WITH THE INSTITUTION OF STRUCTURAL ENGINEERS.

11 February, 1937.

Sir GEORGE HUMPHREYS, K.B.E., Past-President Inst. C.E., President, British Section, Société des Ingénieurs Civils de France, in the Chair.

"The Construction of Large Modern Water Dams."

By MONSIEUR A. COYNE, Ingénieur-en-Chef des Ponts et Chaussées.

ABRIDGED REPORT.[1]

Monsieur Coyne said that up to 10 years ago it was the French practice to erect gravity dams in masonry, although experienced engineers had suggested arched dams whose stability was effected by shape rather than by weight. The weight, however, was by no means a negligible quantity and added considerably to the strength of the work, in spite of the apparently light form of construction. Eventually confidence was created in the arched dam, partly as the result of American research, and that form of design was adopted.

Although the calculations for an arched dam were complicated, it was possible to obtain satisfactory results from approximate rules which were simple and erred on the safe side. Certain points were, however, not easy to solve without actual experience. One was the limiting ratio between the width of the valley and the height of the dam. There were several arched dams with a ratio of between 3

[1] Published in full in Journal Inst. Struct. E., vol. xv (1937), p. 70. (February, 1937.)

and 4, but Monsieur Coyne said that he had decided to exceed that figure in the St. Etienne Cantales dam, where he had adopted a ratio of 6. In another case the ratio adopted was 5.

A further point that had to be taken into consideration was the effect of fixing the arch at the bottom of the valley, as compared with that of fixing it at the sides. Monsieur Coyne said that he had decided that the correct solution was to reduce the thickness at the base so as to make the structure more flexible in a vertical direction, If the dam were fixed too securely at the bottom, that would result in producing excessive strains, usually shown by horizontal cracks upstream. In the Marèges dam, which was 90 metres high, there was a maximum over-hang of 7 metres, as the result of reducing the thickness at the base. He pointed out, further, that the rising of the water on the upstream side caused considerable stresses in the base, and, in order to reduce their effect, he had arranged for the arch to be supported at its base through the intermediary of large concrete props cast with it. A dry joint was provided between the props and the base to allow the props to rise slightly from their supports when the retained water reached its highest level.

As he considered that it was very necessary to find out whether any movement really took place, a special stress-detector of acoustic type was designed, consisting of a vibrating wire, the two ends of which were attached to the two blocks under test. The wire was encased in a watertight tube, which ensured both the tension and the elasticity of the apparatus. The vibrating frequency of the wire, depending on its length, varied with the stress to which it was subjected. The wire was set vibrating from a distance by means of an electro-magnet and the vibrations were measured electrically at a central point, the pitch of the wire being compared with a standard pitch-control apparatus. The measurements recorded showed that the props had rested continually on their supports, and from that it would appear that most of the hypotheses generally used in calculations for arched dams were erroneous. A large number of those acoustic recorders had also been used throughout the whole arch, and the results showed that the stresses recorded in the arch were similar to, but lower than, those worked out theoretically.

Monsieur Coyne then described certain methods of increasing the stability of dams by means of steel ties through the centre of the dam. In the Cheurfas dam in the Province of Oran, Algeria, vertical holes 25 centimetres in diameter and 50 metres in depth were made through the dam and deep down into the sub-soil. At the lower end they were enlarged into two anchoring compartments, one above the other. The cables were lowered down the holes and

so arranged that the strands at the lower end spread out under the weight of the cable. Cement was then injected at the bottom of the hole by means of a pipe let down at the same time as the cable, and in that manner the cable was permanently fixed in the anchoring compartment. On the crown of the dam the cable strands were spread out and fixed in a reinforced-concrete head. Large jacks, supported by the crown, were used to apply a tension of 1,000 tons to each of the ties. There were in all thirty-seven ties, and it was possible by that means to alter the pressure-curve through the dam. Monsieur Coyne said that that method was capable of being employed in many other cases, and he quoted, as an example, the steps taken to give greater stability to the Jument d'Ouessant lighthouse; he suggested that that method might be used in meeting the thrust of arched bridges and the tension of the cables of suspension bridges.

Monsieur Coyne also mentioned the use of pre-stressed steel in pressure-pipes on the Marèges plant. The method employed was somewhat similar to that used in the barrels of large guns where the radial stresses were met by bands of steel wire. At Marèges, the tunnel for the pipe having been excavated in the usual way, circular tubes were arranged in it at intervals of 0·5 metre to act as cable guides, each tube containing a steel cable to be placed under tension. At the two ends of the horizontal diameter the tube was widened out into the shape of a triangular box in such a way as to allow of sufficient play for the tensioning of the cable. Hydraulic jacks were used to obtain a tension of 135 tons, without cracking the concrete. The resulting compression of the concrete was 1,280 lbs. per square inch.

Monsieur Coyne, in conclusion, referred to works designed for dealing with flood-water discharge. At the Marèges dam that was arranged for by two discharge tunnels on one bank and a spillway of the usual type for a gravity dam near the other bank. As a result of careful tests with small-scale models, the entrances to the discharge tunnels were provided with large funnels, with converging-plane vertical walls, which served to direct the stream of water. The walls were prolonged upstream by large circular nose-pieces, the effect of which was to overcome the lateral contraction. The spillway near the other bank, instead of being prolonged to the foot of the dam as was usually done, stopped short at mid-height. That resulted not only in economy in construction and maintenance costs, but also in the water being projected so as to fall at some distance from the foot of the dam where danger was possible from scour. Monsieur Coyne suggested that it might be possible, in some future scheme, to construct the power-station under the overflow of water, which could

be arranged to fall about 100 metres from the dam; space might therefore be saved.

The Paper was illustrated by lantern-slides and a cinematograph film.

A vote of thanks was proposed by Mr. S. B. Donkin, Vice-President Inst. C.E., and seconded by Lt.-Col. H. S. Rogers, C.M.G., D.S.O., Vice-President Inst. Struct. E., and the Meeting concluded with a short discussion in which Messrs. W. T. Halcrow, J. S. Wilson, and James Williamson took part.

Paper No. 5087.

"Flood-Hydrographs."

By Bertram Darell Richards, B.Sc. (Eng.), M. Inst. C.E.

(Ordered by the Council to be published with written discussion.)[1]

TABLE OF CONTENTS.

PART I.

Introduction.

In problems affecting the regulation of floods it is not sufficient to know only the maximum intensity of flood that may be expected; it is necessary also to have curves of its rise and subsequent fall. The maximum intensity and period of rise are intimately connected, and the object of this Paper is to put forward formulas for the calculation of each and for the design of the complete flood-hydrograph for any catchment.

The factors affecting floods are so varied and complex that no precise mathematical computation is possible, and actual records, wherever available, are preferable to any estimate that can be made.

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[1] Correspondence on this Paper can be accepted until the 15th July, 1937, and will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

Major floods from any catchment may, however, occur at long intervals and, in the absence of adequate data, it may be necessary to rely on estimated figures. Suitable formulas for such a purpose should include coefficients for the principal factors affecting the flood, and, in dealing with any catchment, values should be assigned to these coefficients to meet local conditions, such values being based both on analogy to other catchment-areas and on experience.

The more important factors affecting floods are :—

(1) Intensity and duration of the rainfall.
(2) Area of the catchment.
(3) Shape of the catchment.
(4) Steepness of the catchment.
(5) Nature of the surface.
(6) State of wetness of the catchment prior to the rain causing the flood.

The factor "nature of the surface" includes such characteristics as porosity of the soil, tree-growth and herbage, and such points generally as affect the immediate yield of the catchment.

Methods of Flood-Estimation.

The methods of flood-estimation set out in this Paper provide for each of the above factors. The fundamental principles embodied are

(i) that the rainfall-intensity is an inverse function of its duration; and
(ii) that the average intensity of rainfall over an area in relation to the maximum rainfall is an inverse function of that area;

and the following assumptions are made :—

(i) that the rainfall is uniformly distributed over the catchment;
(ii) that the maximum flood occurs when the whole catchment-area is contributing.

It follows that the maximum intensity of flood will occur when the storm is concentric with the catchment-area and has a duration equal to the period of concentration of the flood.

The Institution has appointed a Committee on Floods in Relation to

Reservoir Practice,[1] and from a study of British rainfall records they deduced an expression for the relation of I, the maximum intensity of rainfall in inches per hour, to T, the duration in hours, in which $I = \frac{8}{T+1}$ for abnormal British rainfall and $I = \frac{4}{T+1}$ for rainfall likely to produce normal floods.

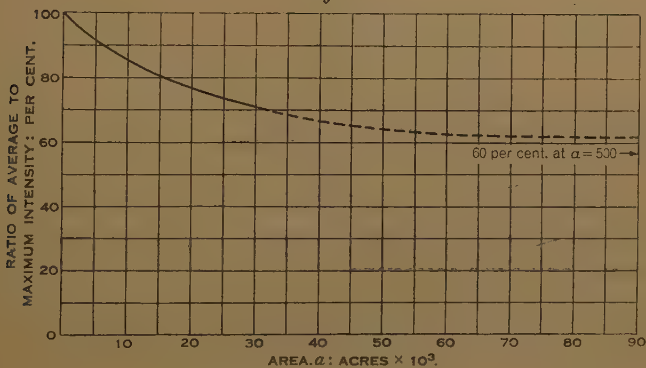
This formula has been adopted in its general form

$$I = \frac{R}{T+1},$$

where R denotes a rainfall-coefficient.

The Committee also gave a curve based on Dr. John Glasspoole's data, connecting the ratio of average to maximum intensity of rainfall

Fig. 1.



with the area affected. This curve is reproduced in *Fig. 1.* As it only applies to areas up to 50 square miles, it has been extended as a dotted line, in a manner which would appear to be borne out by Dr. Glasspoole's investigations. This curve is based entirely on British records and it is possible that it might vary for continental or other areas. It has therefore been represented by a general equation

$$\frac{i}{I} = f(a)$$

in which i denotes the average intensity of rainfall in inches per

[1] Inst. C.E., 1933.

hour, I denotes the maximum intensity of rainfall, and a denotes the area of the catchment in units of 1,000 acres.

Then

$$i = I.f(a)$$

and inserting the value of I, this becomes

$$i = \frac{R.f(a)}{T+1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Period of Rise of the Flood.

Assuming that the rainfall is general over the whole catchment and that the furthest point of the catchment is at a distance L from the reservoir or point of concentration of the flood, the flood will rise from 0 to a maximum in time t, the time taken by the water to travel the distance L.

If v denotes the velocity of run-off,

$$t = \frac{L}{v}.$$

Now considering the water as running in a thin sheet from the catchment,

$$v = \sqrt{ds},$$

where d denotes the depth of the water, s denotes the slope of the catchment, and c is a coefficient.

But $$d = Kit,$$

where K denotes a run-off coefficient.

Therefore

$$t = \frac{L}{v} = \frac{L}{c\sqrt{Kits}},$$

whence $$t^3 = \frac{CL^2}{Kis}, \text{ where } C = \frac{1}{c^2}.$$

Now if the duration of the rainfall T equals t, then inserting the value of i from equation (1)

$$t^3 = \frac{CL^2(t+1)}{KsR.f(a)},$$

whence

$$\frac{t^3}{t+1} = \frac{CL^2}{KsR.f(a)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Maximum Intensity of Flood.

The maximum flood will be reached when the whole catchment-area is contributing. The maximum flood will be equal to Kai, and since a run-off of 1 inch per hour from an area of 1,000 acres equals 1,000 cusecs (approx.) the maximum flood-intensity may be expressed in cusecs per 1,000 acres as

$$Q_m = 1{,}000\ Ki \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

There will be a limit to the maximum intensity of flood, which will be referred to later.

Growth of the Flood.

At any time t_1 the flood Q_1 will be that produced by an area a_1 intercepted by an arc of radius r (*Fig. 2*).

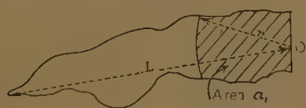
Hence $$Q_1 = Q_m \frac{a_1}{a} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Now $$t_1 = \frac{r}{v_1} = \frac{r}{c\sqrt{Kit_1 s}}, \text{ or } t_1^3 = \frac{Cr^2}{Kis}.$$

But $$t^3 = \frac{CL^2}{Kis},$$

and therefore $$t_1 = t\sqrt[3]{\frac{r^2}{L^2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Fig. 2.



The effect of keeping C as a constant during the growth of the flood will be that the curve of $Q:t$ will be slightly steeper at the commencement than it should be.

The formulas may now be summarized as follows :—

(1) . . . . $i = \dfrac{R.f(a)}{t+1}$

(2) . . . . $\dfrac{t^3}{t+1} = \dfrac{CL^2}{KsR.f(a)}$

(3) . . . . $Q_m = 1{,}000\ Ki$

$$Q_1 = Q_m \frac{a_1}{a} \qquad (4)$$

$$t_1 = t \sqrt[3]{\frac{r^2}{L^2}} \qquad (5)$$

where

t denotes the period of rise of the flood in hours,
t_1 ,, time to reach the intensity Q_1,
Q_m ,, maximum intensity of the flood in cusecs per 1,000 acres,
Q_1 ,, intensity at time t_1,
i ,, average intensity of rainfall over the catchment in inches per hour,
a ,, area of the catchment in units of 1,000 acres,
a_1 ,, area of part of the catchment producing the flood-intensity Q_1,
r ,, radius of the arc intercepting the area a_1,
L ,, distance in miles to the furthest point of the catchment,
s ,, coefficient of steepness or slope,
K ,, run-off coefficient,
R ,, rainfall coefficient,
and C is a coefficient.

Referring to the six important factors affecting floods, mentioned at the beginning of this Paper, it will be seen that all except the last have been provided for :—

(1) The intensity and duration of the rainfall are covered by R and t.
(2) The area of the catchment is covered by a.
(3) The shape of the catchment is covered by the relation of a, a_1 and L.
(4) The steepness of the catchment is covered by the coefficient s.
(5) The nature of the surface is covered by the run-off coefficient K.

Provision will be made later for the sixth, the state of wetness of the catchment.

Unless the catchment is very irregular, it will be convenient to treat it as a rectangle of length l and breadth b (*Fig. 3*). Then if $l : b = n$, and since L is in miles and a is in units of 1,000 acres,

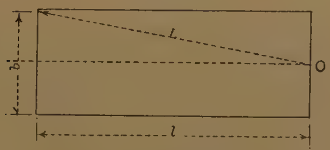
$$L^2 = \frac{a}{0{\cdot}64} \times \frac{4n^2 + 1}{4n} \qquad (6)$$

In the series of equations, the variables are therefore R, a, s, K and n, and the effect on Q_m and t of varying each is as follows :—

Variable.	$\frac{t^3}{t+1}$ varies as	Q_m varies as
R	$\frac{C^*}{R}$	$\frac{R}{t+1}$
a	$\frac{a}{0 \cdot 64 . f(a)}$	$\frac{f(a)}{t+1}$
s	$\frac{1}{s}$	$\frac{1}{t+1}$
K	$\frac{C^*}{K}$	$\frac{K}{t+1}$
n	$\frac{4n^2+1}{4n}$	$\frac{1}{t+1}$

* Since C is a function of KR. (See later.)

Fig. 3.



The relative effect on Q_m and t of the variation of each of these factors, the others being kept constant, is shown in the series of curves in *Figs. 4* and *5* (pp. 412 and 413). A standard catchment has been assumed for purposes of comparison, whose factors are :—

$$a = 16{,}000 \text{ acres}, \; n = 1 \cdot 67, \; K = 0 \cdot 6, \; s = 0 \cdot 03, \; R = 4.$$

The factors a and n are determinable from the survey of the catchment.

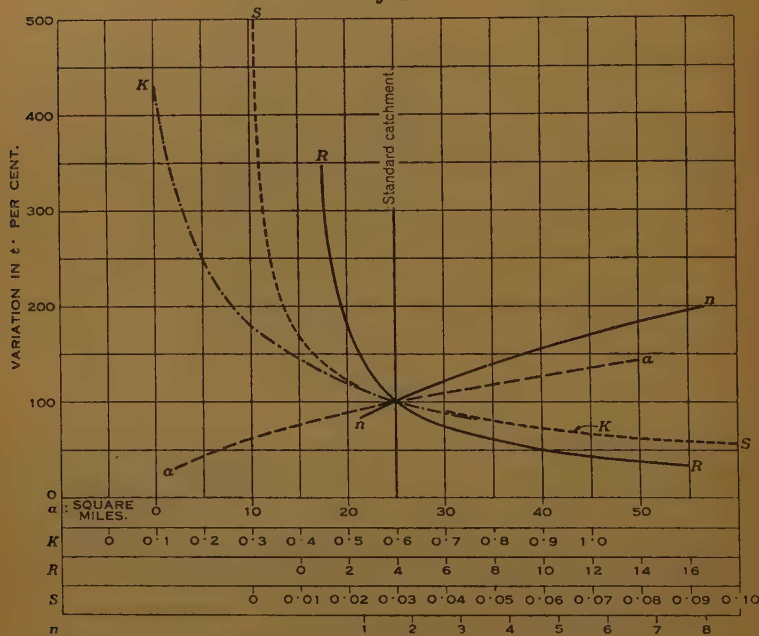
The factor s is a coefficient of steepness, and must be determined from the survey and inspection of the catchment. The Institution Committee on Floods in relation to Reservoir Practice [1] consider that a value of 0·03 is an average for British upland catchments. It is suggested that 0·03 might be taken as an average value, whilst 0·001 for a very flat catchment and 0·10 for a very steep one would represent the probable extremes for most catchments. It will be seen from *Fig. 5* that, in the case taken, these extremes would give

[1] Footnote 1, p. 407.

Q_m values of 24 per cent. and 147 per cent. of that for the average value of $s = 0{\cdot}03$.

The factor R is based on the heaviest rainfall likely to occur, as deduced from actual records of the catchment or from those of neighbouring or similar catchments. A rainfall of high intensity and short duration may give a lower value for R than a less intense but more prolonged fall, and the highest value found should be adopted. The Institution Committee has, as already stated, pro-

Fig. 4.



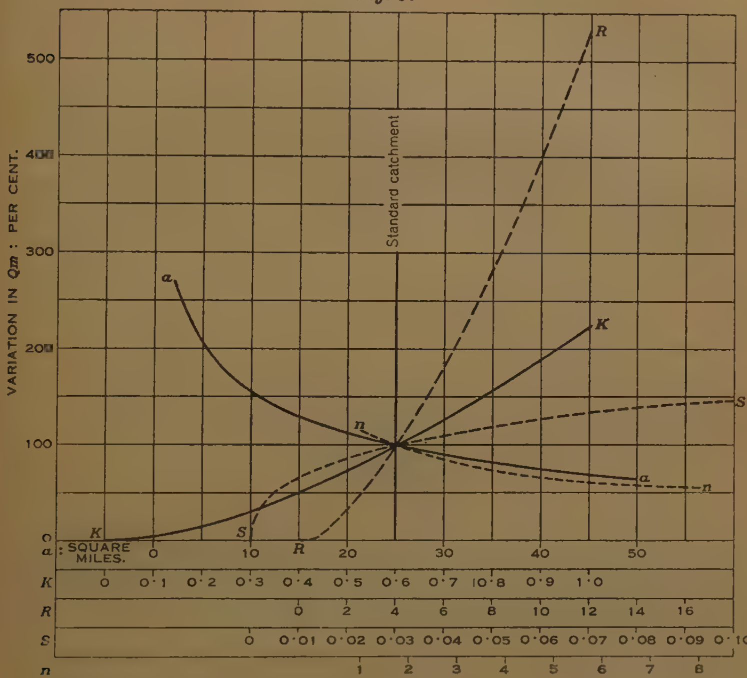
posed for normal British floods $R = 4$, and for abnormal or catastrophic floods $R = 8$. Much higher values would undoubtedly be found in other parts of the world. Records of rainfall quoted in Lacey's "Hydrology and Ground Water,"[1] give, for instance, an equivalent value for R of 17·33 for a storm in California, and of 31 for a cyclonic storm in Gujerat, India, where 30 inches of rain fell in 30 hours.

The factor K represents the immediate run-off factor or the percentage of the run-on passing from the catchment as surface flow. The difference $(1 - K)$ represents loss and ground-water.

[1] J. M. Lacey. London, 1926.

Part of this ground-water may reappear as springs and find its way to the reservoir. It will, however, be unlikely to affect the flood except to the extent of flattening out the lower part of the curve of falling flood in the hydrograph. K will depend on a variety of conditions, such as porosity of the soil, steepness of the ground, tree-growth and herbage, etc. Assuming that its correct value for a certain catchment is 0·6, an incorrect assessment at 0·5 or 0·7 would, from *Fig. 5*, give Q_m at 73 per cent. and 125 per cent. of the

Fig. 5.



correct value respectively. Strictly speaking, K does not remain constant during a flood, but gradually increases as a function of it. In taking it as constant, an average value is being taken. If it could be assigned a variable value, the effect on the flood-hydrograph would be to flatten out the lower part of the curve of rising flood, and to steepen the upper part. In relation to the lag-effect of a reservoir, this modified hydrograph would be less exacting, and therefore in this respect the fact of keeping K as a constant is on the safe side.

The factor n will only appear where the catchment is sufficiently regular in shape as to be treated as a rectangle. The Institution Committee state that for British upland catchments, the ratio $L : D = 1{\cdot}2$ generally obtains, where D denotes the diameter of a circle of area equal to that of the catchment. This is for all practical purposes equivalent to $n = 1{\cdot}67$.

Coefficient C.

The value of C is given by $C = \frac{m}{c^2}$, where m denotes a numerical constant depending on the units taken, and c denotes Kutter's coefficient, which is a function of $\sqrt{d}$, s and N. For value of s greater than 0·001, c is unaffected by s, and if N, the coefficient of roughness (which is to some extent provided for by the run-off coefficient K), is treated as a constant, then c is a function of $\sqrt{d}$ only. With an increase of d, c increases and C decreases. C can be calculated for each case by a method of approximation, but this can be simplified:

$$d = KR \,.\, f(a) \,.\, \frac{t}{t+1}.$$

With an increase of a, $f(a)$ decreases and t increases.
,, ,, KR, t decreases.
,, ,, t, $\frac{t}{t+1}$ increases.

The variation of d, and hence that of C, is thus automatically restricted, but not to the extent that C can be treated as a constant.

It is found, however, that C can be made a function of KR only without introducing any appreciable error into the estimate of Q. The following values of C may be taken for various values of KR, intermediate values being found by interpolation, or more correctly by plotting the curve.

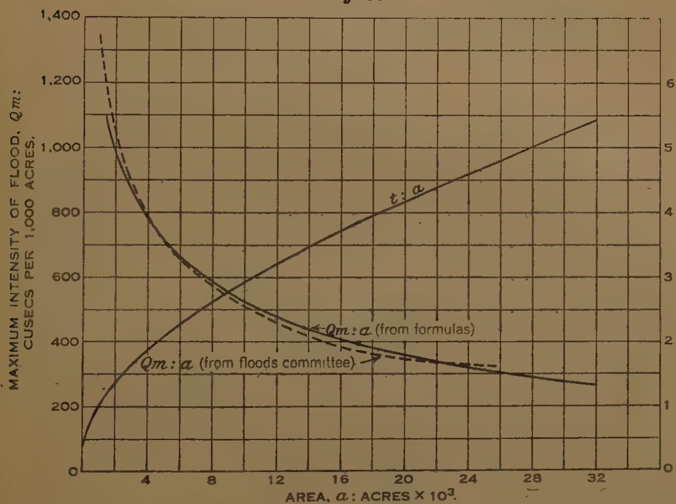
KR.	C
0·6	0·050
1·2	0·025
2·4	0·014
4·8	0·0090
7·2	0·0067
9·6	0·0056

Relation of Maximum Intensity of Flood to Catchment-Area.

In *Fig. 6* are given curves for Q_m and t in relation to a, the catchment-area, for areas up to 50 square miles. The conditions taken are $R = 4$, $K = 0{\cdot}6$, $s = 0{\cdot}03$ and $n = 1{\cdot}67$.

For comparison the curve relating Q_m to a is reproduced as a dotted line, deduced by the Committee from actual records of British floods and considered by them to represent normal floods to be

Fig. 6.



expected on British upland catchments, whose characteristics they give as $R = 4$, $s = 0{\cdot}03$ and $L : D = 1{\cdot}2$ (equivalent to $n = 1{\cdot}67$).

The two curves will be seen to be in close agreement.

Flood-Hydrographs.

The flood-hydrograph as deduced from the formulas will show a flood-intensity rising at a gradually increasing rate, from 0 to a maximum, when it will immediately commence to fall in a similar curve. The total duration of the flood will be twice that of the period of concentration.

Typical hydrographs are shown in *Fig. 7* (p. 416) in which $R = 4$, $K = 0{\cdot}6$, $s = 0{\cdot}03$. As has already been mentioned, the true hydrograph should show the lower parts of the curves somewhat flatter and the top parts correspondingly steeper as a result of C and K having

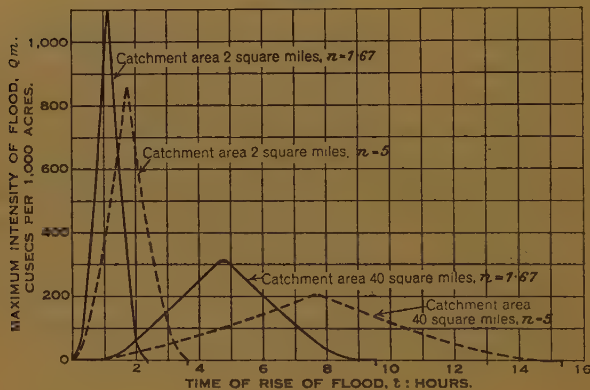
been treated as constant during the progress of the flood. This error is, however, on the safe side. Another modification is the possible tailing away of the falling curve due to the reappearance of ground-water. As the upper part only of the falling-flood curve affects reservoir lag, this modification is of no practical importance.

The flood-intensity can only remain at a maximum for any period under either of two conditions :—

(1) If there is an initial flow when the rain causing the flood commences.

(2) If the rainfall intensity is less than i.

Fig. 7.



Considering the first condition, let there be an initial flood-intensity Q_o when the storm causing the main flood commences. The time which the main storm would have taken to raise the flood from 0 to Q_o will have been saved and the maximum flood will be reached at a time t_o before the cessation of the rain. The flood will therefore continue at its maximum intensity Q_m for a period t_o, after which it will commence to fall as before.

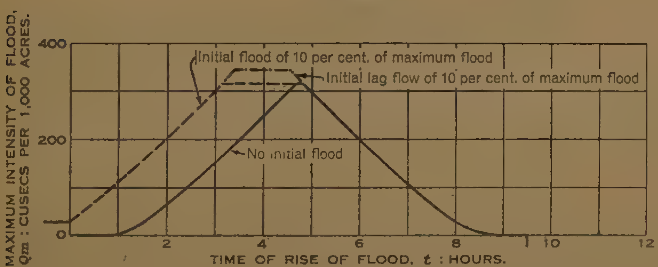
This is shown in *Fig. 8* for an initial flood $Q_o = 10$ per cent. of Q_m. This implies that the initial flood was caused by continuous rain falling at the time when the main storm commenced, and, being superimposed on this initial rainfall, raised the flood intensity to Q_m. If, however, this initial flow was a lag flow and not due to rain actually falling at the time when the main storm commenced, then the flood-intensity Q_m due to the main storm will be superimposed upon the lag flow Q_o to give a total flood-intensity of $Q_m + Q_o$. The time t_1 taken for the flood to grow from Q_m to $Q_m + Q_o$ will be less than t_o and the maximum flood will continue for a period of

$t_0 - 2t_1$. This is also shown in *Fig. 8*, which gives comparative hydrographs for :—

(1) No initial flood.
(2) Initial flood of 10 per cent. of Q_m.
(3) Initial lag flow of 10 per cent. of Q_m.

Turning to the second condition under which there may be a prolonged maximum flood, the rainfall-intensity is based on a duration of rainfall equal to the time taken by the water to travel from the furthest point on the catchment. If a lower intensity occurs, the equivalent duration of the rainfall would exceed this time ; the rate of rise will be slower and the maximum flood less, but it will continue for a period at maximum.

Fig. 8.



This is shown in *Fig. 9* (p. 418) in which $a = 40$ square miles, $K = 0{\cdot}6$, $S = 0{\cdot}03$, $R = 4$, and $n = 1{\cdot}67$. In the case taken the value of i is 0·505 inch per hour and the modified flood-hydrograph is that for a reduced intensity of 0·40 inch per hour.

Limit of Maximum Flood-Intensity.

The maximum flood-intensity in cusecs per 1,000 acres $= Q_m = 1{,}000\ Ki$, where

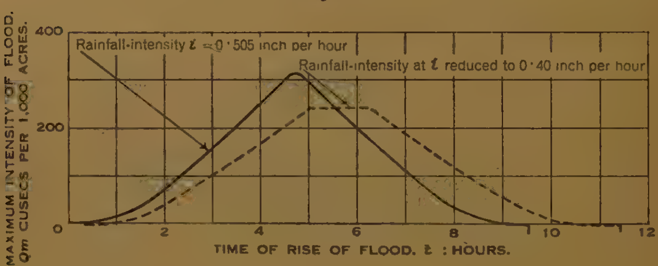
$$i = I\,.f(a) = \frac{R\,.f(a)}{t+1}.$$

In a very small steep catchment, $f(a)$ will approach unity and t will become very small ; i will thus become large, and at the limit it will equal R. Now the highest values of R are derived from periods of heavy prolonged rainfall, and, as already shown, they may reach values of 30 or more. It is, therefore, possible to get the anomalous result of a heavy and prolonged rainfall on a small catchment, producing a rapid and very intense peaked flood. Actually, such a rainfall must in such a case produce a quick-rising but prolonged

flood, whose maximum intensity will be measured by the maximum intensity of rainfall over the period t. This is clearly a case where the hydrograph will be modified, as described in the last paragraph, to the form shown in *Fig. 9*.

In any practical catchment, the value of t, the period of concentration, will be appreciable, and as a minimum it may be put at, say, 0·5 hour. It is noticeable that, whilst prolonged heavy rainfalls are peculiar to the tropics, very intense falls of short duration are met with in most countries, and even in normally dry areas. From a

Fig. 9.



study of the comprehensive list of intense rainfalls of short duration given as an Appendix to Sir Gordon Hearn's Paper,[1] the following values are taken as the extremes covering most cases, and possible in most countries.

For 0·5 hours,	$I = 6$	inches per hour,	giving	$R = 9$
„ 1·0 „	$I = 5$	„ „	„	$R = 10$
„ 3·0 „	$I = 3$	„ „	„	$R = 12$

From these, the maximum average intensity of rainfall over the catchment is deduced, and is $i_1 = I.f(a)$.

Such short intense rainfall gives comparatively low values of R. High values are given by prolonged intense rainfall as is met with in the tropics.

Now let i denote the rainfall-intensity, and let t denote the period of concentration of the flood as calculated from the formulas. If i is found to exceed the maximum intensity for the corresponding period, as given above, then i_1 should be taken as the maximum intensity, and the corresponding period of concentration will be $t_1 = t\sqrt[3]{\frac{i}{i_1}}$.

[1] "The Effect of Shape of Catchment on Flood-Discharge." Minutes of Proceedings Inst. C.E., vol. ccxvii (1923–24, part I), p. 267.

The duration of the rainfall, T, corresponding to the intensity i_1, is given by

$$i_1 = \frac{R \cdot f(a)}{T+1},$$

and the period of the peak flood will be $(T - t_1)$. A modified hydrograph of the form shown in *Fig. 9* is thus obtained, where $Q = 1{,}000\ Ki_1$, the period of concentration is t_1, and the duration of the peak flood is $(T - t_1)$. This modification will occur where R is very large and the catchment is small and steep. The modification of the hydrograph for the initial flood will be carried out as before.

Condition of Wetness of the Catchment prior to the Flood.

If the catchment is already wet when the rainfall causing the flood commences, the rate of flow of the water, and hence that of the rise of the flood, will be accelerated; the ultimate maximum flood-intensity will be based on the time taken by the water to reach the reservoir from the most distant point of the catchment, and the maximum flood will remain the same but there will be a period of constant maximum. The effect of a wet catchment is therefore equivalent to that of an initial flood.

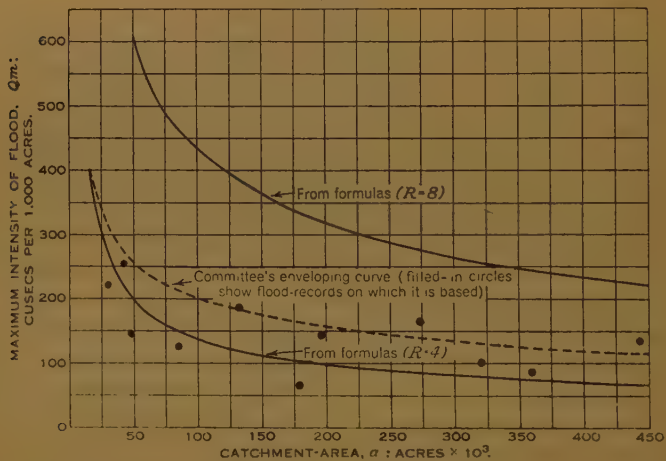
The object being to find the worst condition of flood that may be expected from a catchment, it must for safety be assumed that the storm causing the flood acts on a wet catchment. It is suggested that a reasonable allowance to make for this condition of initial wetness is to assume that rain has already been falling continuously and produced an initial flood of 10 per cent. of the maximum flood-intensity Q_m. Provision for the wetness of the catchment will thus reduce the hydrograph to the form of curve shown dotted in *Fig. 8* (p. 417).

LARGE CATCHMENT-AREAS.

In *Fig. 10* (p. 420) is shown a curve of maximum flood-intensity for areas up to 450,000 acres (roughly 700 square miles), the coefficients being taken as before. For comparison the flood-records from the Committee's report are shown as small filled-in circles, and their enveloping curve is also reproduced. It will be seen that the curve obtained from the formulas, which agreed closely with the Committee's curve for small areas, is well below it for large areas. The points on which the Committee's curve is based represent the records of eleven floods spread over a period of 82 years; their

curve gives a fair average for the upper six points, whilst that obtained from the formulas is an equally good average for the eight lower points. The general form of the two curves is the same. The formula-curve is drawn for a rainfall coefficient $R = 4$, applicable to rainfall causing normal floods. For rainfall such as would produce abnormal or catastrophic floods, the value would be nearer $R = 8$. On the same diagram is given a curve for $R = 8$, and it lies well above all the flood-points. The eleven flood-points are very scattered, which suggests either that the conditions represented by the factors K, s and n are not all uniform in these larger catchments to which

Fig. 10.



the records apply, or else that the floods were not in each case attributable to the normal maximum rainfall-intensity of $R = 4$. For comparing the actual flood with that deduced from the formulas, it is necessary to know the coefficient R of the rainfall which produced that flood. Only those points representing floods caused by rainfall of intensity $R = 4$ can be expected to lie on the formula-curve drawn for $R = 4$. If the flood were caused by rainfall of lower intensity, the point would fall below the curve, and if it were higher, it would fall above. Whilst the few flood-points available are insufficient to establish definitely the applicability of the formulas to large catchments, the general form of the curve suggests that they can be so applied.

Check of Result obtained from Formulas by actual Flood-Records.

A major flood can be used to check the results obtained from the formulas, provided that an actual hydrograph of the flood is taken together with a synchronized record of the rainfall at the storm-centre, the position of which should be determined.

Assuming coefficients for the catchment, a theoretical hydrograph is drawn for a rainfall-coefficient based on the actual record, allowance being made for the eccentricity of the storm; this hydrograph is then modified as may be necessary for lag-flow or initial flood (as in *Fig. 8*), and for period of concentration (as in *Fig. 9*), to bring it in accord with the conditions of the actual flood.

Special Cases to which the Formulas do not apply.

The formulas would not apply in countries where the maximum floods result from the thaw in spring of the snow accumulated on the catchment during the winter. They would also require modification in cases where the nature of the catchment was such as to provide a considerable temporary storage, although in some cases this might be allowed for in the assessment of the coefficient K.

Conclusion.

The formulas and methods of estimating floods as set out above are not claimed to be as in any way precise; they involve of necessity too many assumptions, such as uniformity of catchment-slope and of rainfall-distribution. Their practical value lies rather in the fact that they contain variable coefficients for the principal factors affecting floods, which can be given suitable values assessed by analogy to other catchments and by the experience of the engineer. They are, therefore, of wider application than formulas deduced for any special district and not containing all these variables. No flood-formulas, of whatever practical value, obviate the desirability of collecting all possible records of floods and river-flow; this is most systematically carried out in some countries. Formulas and recorded data are complementary: the data enable the values of the coefficients to be determined, whilst the formulas allow of the adjustment of flood-records to other conditions and of their utilization in estimating floods for other catchments where there are less adequate data available.

Flood-records are of most value if they show the continuous dis charge from commencement to finish of the flood, accompanied by a synchronized continuous record of rainfall. If the flood is measured as the outflow of a reservoir, then adjustment to inflow must be made by taking into account the reservoir lag.

PART II.

Determination of Reservoir Lag-Effect.

It is now necessary to deal with the effect of a flood on a storage reservoir, and a method is described below by which the essential results can be very quickly determined; these are the maximum height to which the water-level of the reservoir will rise with a given length of waste-weir, or, conversely, the length of weir necessary for any permissible rise, and the time from the commencement of the flood until the maximum height will be reached.

A typical hydrograph is shown in *Figs. 11.* The flood rises from an initial intensity Q_o to a maximum intensity Q_m in t_1 hours, remains at Q_m for t_2 hours, and then falls to Q_o again in t_3 hours; the total duration of the flood is thus $t = t_1 + t_2 + t_3$. The dotted curve represents the rate of outflow over the weir. The area of the hydrograph up to a vertical line at any time T represents the total inflow in the time T. The corresponding area below the outflow curve represents the total outflow in the same time, and the area between the two curves represents the water temporarily stored: this is the area of the reservoir multiplied by the rise of level. The total inflow in any time is equal to the outflow plus the water stored.

Where the flood-intensity is constant, as in the section BC of the hydrograph (where it is Q_m), the relation of rise of reservoir-level to time is represented by the differential equation

$$Q_m dt = a.dh + clh^{\frac{3}{2}}.dt,$$

and

$$T_2 - T_1 = \int_{h_1}^{h_2} \frac{a}{Q_m - clh^{\frac{3}{2}}} \, . \, dh,$$

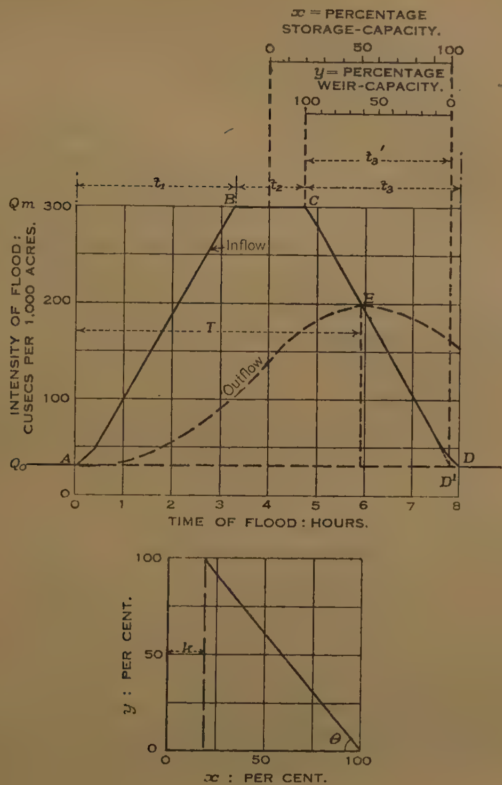
where a denotes the area of the reservoir,

l ,, the length of the weir,

h ,, depth of water over the crest of the weir,

c ,, the weir-coefficient.

Figs. 11.



$$K = 100\left(\frac{\frac{1}{2}t_2}{\frac{1}{2}t_2 + t_3}\right)$$

$$\tan\theta = \frac{100}{100-K} = \frac{t_2+2t_3}{2t_3}$$

$$y = (100-x)\tan\theta$$

This enables the ratio of rise to time to be determined for a flood of constant intensity. During the periods of rise and fall of the flood, Q, the intensity, is a function of the time, and the relation of rise to time is represented by the differential equation

$$f(t).dt = clh^{\frac{3}{2}}.dt \pm a.dh.$$

This equation is intractable, as the variables cannot be separated. The lag-effect can, however, be calculated by a step-by-step method. The hydrograph may be divided into vertical strips representing short intervals of time, and the inflow, outflow, and water stored to the end of each interval can be determined. This method is de-

scribed in detail in the Interim Report of The Institution Committee.[1] This method is accurate if the intervals of time are made sufficiently small, but it is long, particularly where the length of weir for a given level of water has to be determined. A shorter method is here put forward.

Referring to *Figs. 11*, the outflow curve cuts the inflow curve at a point E. At this point the inflow is equal to the outflow and the reservoir level is at its maximum. The position of E depends on the storage; that is, on ah, where h is the rise of level of the reservoir. For any storage there will be a corresponding length of weir. The method to be described is based on the determination of the position of the point E. If there were no storage, that is, if $a = 0$, the inflow and outflow curves would coincide. The original level of the reservoir would be h_o and the maximum level would be h_m, these values being such that

$$Q_o = clh_o^{\frac{3}{2}},$$

and

$$Q_m = clh_m^{\frac{3}{2}}.$$

The maximum level h_m would be reached at $T = t_1$, would remain thus until $T = t_1 + t_2$, and would then fall again to h_o at $T = t_1 + t_2 + t_3$. If there were no initial flood, then t_2 would be zero and the hydrograph would rise to a peak at t_1. (See p. 416.)

The middle point of the hydrograph, that is, where $T = t_1 + \frac{1}{2}t_2$, represents the point corresponding to zero storage.

Now if the storage were sufficient to accommodate the whole inflow in excess of the initial flood, that is, if A = area $ABCD$ in *Figs. 11*, the maximum level of the reservoir would be reached at $T = t_1 + t_2 + t_3$, and would be equal to $h_o + \frac{A}{a}$ above crest-level. The length $\frac{1}{2}t_2 + t_3$ may therefore be taken as a scale representing storage-capacity from 0 to 100 per cent. The available storage is $a(h - h_o)$ and the full storage is A, and therefore the percentage storage-capacity, x

$$= 100 \cdot \frac{a(h - h_o)}{A} \quad \ldots \ldots \ldots \quad (1)$$

When the weir is discharging the full inflow, its capacity is $Q_m - Q_o$ in excess of Q_o, and at the level h, its capacity is $clh^{\frac{3}{2}} - Q_o$ in excess of Q_o; therefore the percentage weir-capacity, y

$$= 100 \left[\frac{clh^{\frac{3}{2}} - Q_o}{Q_m - Q_o} \right] \quad \ldots \ldots \ldots \quad (2)$$

[1] *Loc. cit.*

For 100 per cent. storage the weir-capacity = 0, and for 100 per cent. weir-capacity, the reservoir remains at a maximum level to the point C, where $T = t_1 + t_2$.

The length t_3 therefore represents the weir-capacity scale from 100 per cent. to 0 per cent., and 100 per cent. on the weir scale corresponds to $\dfrac{100 \times \frac{1}{2}t_2}{\frac{1}{2}t_2 + t_3}$ on the storage scale. Plotting the co-ordinates x and y, it will be seen that

$$y = (100 - x) \tan \theta, \text{ where } \tan \theta = \frac{2t_3 + t_2}{2t_3} \quad . \quad . \quad (3)$$

Expressions are thus available for x, y, and $x : y$, from which h or l, and T, the time to reach the maximum level, can be determined as follows.

Determination of h.—Given a length of weir l, the method of determining the maximum height above the crest, h, to which the water-level will rise, is :—

$$x = 100 \times \frac{a(h - h_o)}{A},$$

$$y = 100 \times \frac{clh^{\frac{3}{2}} - Q_c}{Q_m - Q_o},$$

$$y = (100 - x) \times \frac{t_2 + 2t_3}{2t_3}.$$

But a and l are known, t_1, t_3, Q_m, Q_o, and A are found from the hydrograph, and h_o is given by $Q_o = clh_o^{\frac{3}{2}}$.

The equations therefore reduce to the form

$$x = 100(C_1h - C_2),$$
$$y = 100(C_3h^{\frac{3}{2}} - C_4),$$
$$y = C_5(100 - x),$$

where C_1, C_2, etc., are constants.

Inserting the values of x and y from the two first equations in the last equation, a resulting equation is obtained in the form

$$h^{\frac{3}{2}} \times C_6h = C_7,$$

from which h can be found.

The value of x is then found, and thus

$$T = t_1 + \tfrac{1}{2}t_2 + (\tfrac{1}{2}t_2 + t_3) \cdot \frac{x}{100}.$$

In the method described above, a slight correction is necessary. Referring to the diagram it will be seen that the portion ED of the

falling side of the hydrograph has actually no effect on the reservoir lag, and it would make no difference whether it were in line with CE or whether it tailed off slowly. In the equations above, however, it has a slight effect on h and a very considerable one on T. A correction is therefore necessary so that this tail does not come into play. In the hydrographs designed from the formulas in Part I of the Paper, the line CD approaches nearly to a straight line for a large part of its length, but tails off at the bottom. Where Q_o is 10 per cent. of Q_m this tailing portion is nearly cut off. If the straight line (or very flat curve) of the upper part of CD is continued to cut the base AD at D^1, as shown by the dotted line in *Figs. 11*, a hydrograph will be obtained without any tail on the falling-flood side. The inflow of the corrected hydrograph in excess of Q will now be A = area $ABCD'$, and t_3 will now become t_3', the projection of CD'.

Hence $$\tan\theta = \frac{t_2 + 2t_3'}{2t_3'},$$

and $$T = (t_1 + \tfrac{1}{2}t_2) + \frac{x}{100}(\tfrac{1}{2}t_2 + t_3').$$

Where the ratio of Q_o to Q_m is 10 per cent. or more, this correction makes very little difference, but where it is less, the effect on T is considerable; h is very slightly affected, as the additional area of the tail of the hydrograph is small whilst the difference between t_3 and t_3' is fairly large. In the above equations, therefore, A should be taken as the area $ABCD'$ and t_3 as t_3', the projection of CD'. *Example.*—The application of these equations to the determination of the reservoir lag-effect may be illustrated by a numerical example.

Taking the hydrograph in the diagram :—

$Q_m = 300$ and $Q_o = 30$ cusecs per 1,000 acres of catchment.

Also, $t_1 = 3{\cdot}25$, $t_2 = 1{\cdot}50$, and $t_3' = 3{\cdot}05$ hours, whilst A = area $ABCD'$ = 4,567,000 cubic feet; c, the weir-coefficient, is taken as 3·33, and a, the area of the reservoir, is taken as 5 per cent. of the catchment = 50 acres per 1,000 acres of catchment = 2,178,000 square feet.

(1) *Given l, the length of weir* (= 30 *feet per* 1,000 *acres*), *to find h and T.*

Now $Q_o = clh_o^{\frac{3}{2}} = 30$, and $3{\cdot}33 \times 30h_o^{\frac{3}{2}} = 30$, whence $h_o = 0{\cdot}45$ feet,

and $x = 100 \times \dfrac{a(h-h_o)}{A} = 100 \times \dfrac{2{,}178}{4{,}567}(h - 0{\cdot}45) = 47{\cdot}7(h - 0{\cdot}45)$,

while $$y = \frac{100 \times clh^{\frac{3}{2}} - Q_o}{Q_m - Q_o} = 100 \times \frac{3{\cdot}33 \times 30h^{\frac{3}{2}} - 30}{300 - 30} = \frac{100h^{\frac{3}{2}}}{2{\cdot}7} - 11{\cdot}11,$$

and y also $= (100-x)\tan\theta = (100-x)\dfrac{t_2+2t_3'}{2t_3'} = (100-x)\dfrac{1{\cdot}50+6{\cdot}10}{6{\cdot}10}$.

$$= (100 - x)\ 1{\cdot}246.$$

Then $$\frac{100h^{\frac{3}{2}}}{2{\cdot}7} - 11{\cdot}11 = 1{\cdot}246\ \{100 - 47{\cdot}7(h - 0{\cdot}45)\}$$

whence $$h^{\frac{3}{2}} + 1{\cdot}604h = 4{\cdot}386 \text{ and } h = 1{\cdot}54 \text{ feet.}$$

Hence $x = 47{\cdot}7(h - 0{\cdot}45) = 47{\cdot}7\ (1{\cdot}54 - 0{\cdot}45) = 52{\cdot}0$ per cent.,

and $T = (t_1 + \tfrac{1}{2}t_2) + \dfrac{x}{100}(\tfrac{1}{2}t_2 + t_3') = 3{\cdot}25 + 0{\cdot}75 + \dfrac{52}{100}(0{\cdot}75 + 3{\cdot}05)$

$$= 5{\cdot}98 \text{ hours.}$$

It should be noted that the step-by-step method gives $h = 1{\cdot}56$ foot, and $T = 5{\cdot}9$ hours.

(2) *Given $h = 2$ feet, to find l, the necessary length of weir and T.*

Then $$x = 100 \times \frac{2{,}178}{4{,}567}(2 - h_o) = 95{\cdot}4 - 47{\cdot}7h_o,$$

and $$y = 100 \times \frac{3{\cdot}33 \times l \times 2^{\frac{3}{2}} - 30}{300 - 30} = 3{\cdot}491l - 11{\cdot}11.$$

Also $$y = (100 - x)\tan\theta = 1{\cdot}246\ (100 - x).$$

Hence $$3{\cdot}491l - 11{\cdot}11 = 1{\cdot}246\ (100 - 95{\cdot}4 + 47{\cdot}7h_o),$$

whence $$l - 17{\cdot}02h_o = 4{\cdot}82.$$

Now $Q_o = clh_o^{\frac{3}{2}} = 30$, so $lh_o^{\frac{3}{2}} = 9$, and with one or two approximations l is found to be $16{\cdot}3$ feet, which is the necessary length of weir per 1,000 acres of catchment.

The method described above gives a very quick means of determining the length of weir for any given permissible height of water above the crest (and, conversely, the maximum level to which the reservoir will rise for any given length of weir), and the time from the commencement of the flood at which this maximum level will be reached. The actual curve of rise is of little importance. The method is applicable to flood-hydrographs of a form such as that shown in Part I of this Paper, where the upper part of the curve of falling flood approximates to a straight line.

As a test of the relative accuracy of the method, comparative results have been worked out by this method and by the step-by-step method, using in the latter short intervals of time to give as correct a result as possible. A variety of cases has been taken, using different hydrographs, varying the area of the reservoir, the length of the weir, and the initial flood Q_o. It should be noted that varying

the relation of $Q_o : Q_m$ alters the time t_2, the period of maximum flood on the hydrograph.

Period of the Maximum Flood on the Hydrograph.

The comparative results of the two methods are shown in the Appendix (p. 430). For each case are given the percentage storage, the percentage error in h as calculated by the new method, and the percentage lag-effect. The percentage initial storage is also shown: it is the position of the point C relative to the storage scale, which is governed by the relation (*Figs. 11*, p. 423), of t_2 to t_3'. (In the hydrograph shown it is nearly 20 per cent.)

A study of the Appendix shows that:—

(1) Where the percentage storage is so low as to approach to the percentage initial storage, the percentage lag-effect is too small to be of practical value.

(2) Where the percentage storage is in excess of the percentage initial storage the percentage error in h is small; for $Q_o = 10$ per cent. of Q_m, it is less than 5 per cent., and for $Q_o = 0$, it reaches 7 per cent.

From this it is deduced that for practical cases of reservoir lag-effect, the new method gives values of h within a reasonable limit of accuracy, the error in h compared to the net rise of water-level $(h - h_o)$ being of the order of 5 per cent. From the Appendix it will be seen that the time T to reach the maximum level, as calculated by the new method, corresponds very closely with that found by the step-by-step method.

Effect of Reservoir-Area.

The size of the reservoir necessary to have an appreciable lag-effect is also indicated by the results in the Appendix.

(1) Where $Q_o = 10$ per cent. of Q_m, a reservoir of area less than 2 per cent. of the catchment-area has no practical lag-effect, for a catchment of 40 square miles. With a smaller catchment and corresponding greater flood-intensity, the lag-effect is greater and appreciable for areas of reservoir down to 1 per cent. for a catchment of 2 square miles.

(2) As the initial discharge Q is reduced, the reservoir becomes more effective. Where $Q_o = 0$, on a 40-square-mile catchment, a 1 per cent. reservoir has as great a lag-effect as a 2 per cent. reservoir for the same catchment but with $Q_o = 10$ per cent.

Variation of Reservoir-Area with Height above Crest of Weir.

The reservoir-area has been assumed constant. It would increase with the depth over the crest of the weir, but within the limits of permissible depth the variation of area would generally be unimportant. Where desirable, however, a simple approximate correction can be introduced by substituting for a, the area at crest-level, a', the mean of the areas at crest-level and at maximum level.

The Paper is accompanied by eleven diagrams from which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

Comparison of Results of New Method and of Step-by-Step Method.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Catchment area: square miles.	Q_m	Q_o	Initial storage: per cent.	a: per cent.	l: feet.	h_o: feet.	h_m: feet.	Step-by-step method.		New method.		Storage: per cent.	Error in h: per cent.	Lag-effect: per cent.
	cusecs.							h: feet.	T: hours.	h: feet.	T: hours.			
40	300	30	20	5	10	0·93	4·33	2·44	6·7	2·43	6·72	71·5	0·6	55
40	300	30	20	5	30	0·45	2·08	1·56	5·9	1·54	5·98	52·0	1·8	32
40	300	30	20	2	30	0·45	2·08	1·96	5·0	1·94	5·08	28·4	1·3	7
40	300	30	20	1	30	0·45	2·08	2·05	4·8	2·12	4·60	15·9	—4·4	2
2	1,100	110	20	10	10	2·22	10·28	3·07	1·85	3·08	1·82	95·4	—1·2	89
2	1,100	110	20	5	10	2·22	10·28	3·83	1·75	3·82	1·76	88·6	0·6	80
2	1,100	110	20	2	30	1·06	4·95	3·62	1·40	3·54	1·46	54·8	3·1	34
2	1,100	110	20	1	30	1·06	4·95	4·44	1·25	4·30	1·28	35·8	4·1	13
2	1,100	110	20	0·5	30	1·06	4·95	4·87	1·15	4·90	1·15	21·3	—0·8	2
40	300	100	35	5	30	1·00	2·08	1·79	5·5	1·82	5·35	51·7	—3·8	27
40	300	0	0	5	10	0	4·33	1·46	7·6	1·42	7·46	81·0	2·7	66
40	300	0	0	5	30	0	2·08	1·14	6·7	1·09	6·83	62·2	4·4	45
40	300	0	0	2	30	0	2·08	1·68	5·6	1·56	5·92	35·5	7·1	19
40	300	0	0	1	30	0	2·08	1·92	5·1	1·79	5·43	20·4	6·7	8
40	300	0	0	0·5	30	0	2·08	2·03	4·9	1·93	5·12	11·0	4·9	2

ARRANGEMENT OF COLUMNS IN THE APPENDIX.

Column		
1	Catchment-area in square miles.	
2	Q_m denotes the maximum flood-intensity in cusecs per 1,000 acres.	
3	Q_o ,, initial ,, ,, ,, ,, ,,	
4	Percentage initial storage $= \frac{\frac{1}{2}t_2}{\frac{1}{2}t_2 + t_3'} \times 100.$	
5	a denotes the	area of the reservoir expressed as a percentage of the catchment area.
6	l ,,	length of the weir in feet per 1,000 acres of catchment.
7	h_o ,,	depth over the weir corresponding to Q_o.
8	h_m ,,	,, ,, ,, ,, Q_m.
9	h ,,	maximum height reached, by the step-by-step method.
11	h ,,	,, ,, ,, new method.
10	T ,,	time in hours to reach the maximum height, by the step-by-step method.
12	T ,,	,, ,, ,, ,, ,, new method.
13	Percentage storage.	
14	Percentage error in h by the new method $= \frac{\text{col. (9)} - \text{col. (11)}}{\text{col. (9)} - \text{col. (7)}} \times 100.$	
15	Percentage lag-effect $= \frac{\text{col. (8)} - \text{col. (9)}}{\text{col. (8)} - \text{col. (7)}} \times 100.$	

Paper No. 5049.

"Jetty Repairs on the River Thames."

By Hubert Maurice Gorringe, M.C., B.Sc. (Eng.), Assoc. M. Inst. C.E.

(Ordered by the Council to be published in abstract form.)[1]

The methods described are based on experience gained from repairs which were carried out without interference with the earning capacity of the jetties. These are also of interest in connexion with new construction, in that the results of defective designs are made apparent when repairs become necessary. It has been found that decay becomes first evident in the pitch-pine near the high-water mark, where it is discovered by hammer test for internal decay or with a pricker test for external decay. As soon as external decay is noticed, it can be delayed by cutting out the rot, applying a preservative and inserting graving pieces to prevent water getting to the inside. In due course, however, further repairs will be necessary.

In the jetties under consideration, the first problem was to scarf or "jump" defective piles. Experience has shown that the scarf should be about 5 feet long, and, whenever possible, should be supported by a new strut to support the pile at the weak place against a horizontal blow. The "jumped" joint, on the other hand, is simpler to make and is stronger against horizontal blows. It has two half-timber fish-pieces at least 6 feet long and is preferable to the scarf joint if there is room to construct it. A tenoned pile can be headed with this joint in such a way as to have a standard-sized tenon and yet be made to carry as much weight as the original pile. The cross-cut should be not quite horizontal so that the new pile-head can be pulled into place with stretching-screws. Should the joint become too tight to move further, a saw-scarf will allow extra movement and allow the head to come right home. For this work it is most necessary for all bolts to be a driving fit, and so the scotch augers must be the same size as the bolt. Contractors will resent insistence upon this detail, but it is most important. It has the further advantage that a man can screw up the nut without having to use a spanner on the head as well as on the nut of the bolt. In general,

[1] The MS. and drawings can be seen in the Institution Library.—Sec. Inst. C.E.

plates should be reduced to a minimum on any timber jetty, because they cut and waste the bolts much more than do timber fish-pieces, and the original repair cannot be as good with fish-plates as with fish-timbers. For "jumped" joints fish-plates should never be used.

Diagonals can generally be renewed entirely, but, to prevent low-tide repairs, only the decayed part need be renewed with a butt-cover. This method of construction can be used with advantage for horizontal walings in new jetties as well as for repairing old ones. Halved joints can be avoided entirely whilst timbers need not be all one dead length, and the odd ends can be utilized for the single butt-cover.

Deck beams are generally placed as nearly as possible under the rails, and the joints are frequently scarfed so that the beams shall be in a straight line along the straight portions of the jetty. This design has been found by the Author to be exceedingly unsafe, and a better plan is to put a corbel under the scarf-joint. This joint, however, uses more material than the plain butt-joint. Frequently one length of timber covers three bays, and this allows of breaking the joints on alternate beams.

A big factor in the life of a timber jetty is a good deck of even thickness, with spaces, if possible, between the planks. To prevent water running down on to the beams, $\frac{3}{4}$-inch strips are nailed on each plank as distance pieces, so that rain-water runs down where there is no beam. Vertical fenders require continual repairs, and, for this reason, they should be made of oak and not pitch-pine, which is inclined to split. Oak fenders are found to be quite serviceable on a jetty where barges only are discharged. It is important to make the timber fenders on reinforced-concrete jetties easily renewable, as well as to bed them solidly on the pile or column which they protect.

A rubbing-bar increases the damage in some cases, but it prevents many other cases of damage from being serious. A rubbing-iron must be of heavy section which will not bend and break without a really heavy blow; light bead-iron can be very dangerous. Floating booms can be expensive as, apart from breakages, they wear themselves out in 5 or 6 years. Oregon pine can be used here, and round logs or flat logs are better than square ones which will not float flat in the water.

Where it is found necessary, due to its exposed position, to treat the concrete surface of reinforced-concrete jetties with paraffin-wax or other waterproofing material, some fault in the mixing of the original concrete can be suspected. When faults develop they will take the form of cracks or red patches of rust on the surface of the concrete, the latter showing that water has already penetrated and

caused the steel reinforcement to rust. Later the steel expands due to rust, and cracks form longitudinally in the members. Cracks due to collision or bad design are more frequently tranverse than longitudinal. To repair serious fractures it is necessary to strip the member down to the main steel bars so as to make certain they are not broken. After the bars have been cleaned thoroughly by chipping, and the wire armouring renewed, all the concrete and steel should be coated with a strong cement grout just before casting the new concrete. Vertical piles or columns may require special treatment of the moulds, particularly where an up-joint is required to carry weight at the top of the repair. In the Author's experience the faults on a jetty of indifferent construction, built about the year 1903, were found to be mostly fractures developing from the inside, due to rust on the bars.

The increased cost of repairs on reinforced-concrete jetties, as compared with that on timber jetties, leads many engineers still to favour the timber jetty; other points in favour of timber-jetty construction are :—

1. A timber jetty can be built more quickly.
2. When damaged, the full extent of the damage is easily ascertained, whereas concrete structures may fracture 50 feet away from the point of contact and the fracture be very small to see under water.
3. The initial cost of concrete jetties is greater.
4. A jetty may only be required for temporary work, and, when that work is done, the timber jetty is more easily disposed of.
5. Much more care and supervision are needed when building a concrete jetty. Poor supervision and bad work cannot be put right later without heavy expense.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Joint Sub-Committee on Vibrated Concrete.

In the November, 1935, Journal [1] there was announced the formation, jointly with the Institution of Structural Engineers, of a Sub-Committee of the Research Committee to study the effect of vibration on concrete.

The terms of reference were to consider the best means of investigating the effect of shaking, percussion and vibrating of concrete during deposition, with particular reference to the effect on the setting and ultimate qualities of the concrete. The Chairman of the Joint Sub-Committee is Mr. R. H. H. Stanger and the membership is given in the above-mentioned Journal. There have since been added the following members:—Mr. W. J. E. Binnie, M.A., Mr. W. L. Cowley, Mr. R. E. Holloway, and Mr. W. L. Lowe-Brown, D. Eng., M.Sc.

The investigation falls naturally into two parts, the first part being of a fundamental nature, using an experimental machine capable of a wide range of amplitudes and frequencies and already constructed at the Building Research Station, and the second part comprising full-scale tests of vibrators available on the market. A programme of research extending over 2 years was drawn up, and sufficient financial support having been obtained for the first year's work a start on preliminary laboratory investigations was authorized in January, 1936.

A commencement of the research was made using a small vibration machine [2] capable of vibrating a 50-square-centimetre mould (2·78-inch side). Frequencies between 1,200 and 13,000 revolutions per minute were used with four different amplitudes. The machine gave a circular motion of the mould in a vertical plane. Tests were carried out using a 1-to-2-to-4 concrete with aggregate of $\frac{3}{8}$-inch maximum size and a water/cement ratio of 0·50. These tests indicated that for this particular concrete, for the time of vibration used (2 minutes), there appears to be a critical acceleration independent of frequency, below which the crushing strength falls

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[1] Journal Inst. C.E., vol. 1 (1935–36), p. 42.

[2] An illustration of this machine is given in *Figs. 13* (*c*), p. 339.

off rapidly and above which it increases only slowly; also that, for a given acceleration, there is a critical time of vibration, below which the compressive strength falls off rapidly and above which it shows practically no change.

A larger vibration machine was then constructed and the investigations therewith are now sufficiently advanced to enable an interim report to be issued embodying the results so far obtained.

Investigation on the Vibration of Concrete. Interim Report No. 1. Preliminary Tests.

Details of Machine.

The machine used[1] for the tests consists of a table mounted on flat springs, the lengths of which are adjustable to suit different speeds and weights of concrete, in order that the machine may always operate at resonance. The table is vibrated by an oscillating plunger through a friction-clip, the plunger being driven from a circular cam of adjustable eccentricity, arranged to impart a simple harmonic motion in a vertical plane. The friction-clip drive ensures that the amplitude of the table, although operating at resonance, does not exceed the amplitude for which the cam is set. Any such tendency is checked by the friction force changing its direction and acting as a damping force instead of as a driving force. The frequency[2] of vibration can be varied between 500 and 8,000 vibrations per minute, and the amplitude[2] of the vibration (that is, the displacement from the mid-position) can be varied from 0 to $\frac{1}{8}$ inch with an acceleration[2] up to $10g$.

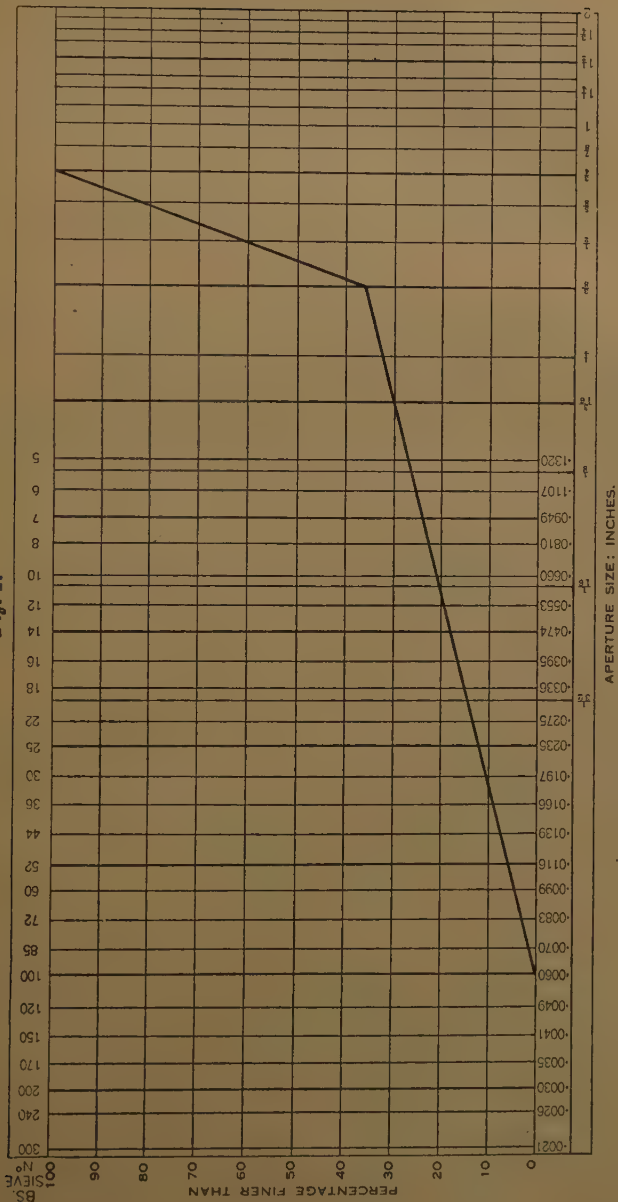
Details of Concrete.

The concrete used for all tests was a 1-to-1·8-to-4·2 mix (by weight) using an ordinary Portland cement and river sand and gravel. Tests at the Building Research Station have shown that, for dry mixes, the sand-content should be reduced for maximum workability, and the above mix was chosen as being the most workable for the lower water-contents used in the present tests. The maximum size of aggregate was $\frac{3}{4}$ inch, and the grading is shown in *Fig. 1*.

The materials used for the tests were kept constant throughout each series of tests, but were not necessarily quite the same for different series. Comparison of results in different series may not, therefore, be fully justifiable. The specimens were 4-inch cubes,

[1] See *Figs. 13 (a)*, p. 339.
[2] The definitions are set out in the Appendix, p. 449.

Fig. 1.



GRADING USED FOR VIBRATION-COMPACTION TESTS.

six similar specimens being cast at the same time. The density of the concrete was measured at 1 day, immediately after demoulding. The cubes were then stored in water until 7 days, when they were tested for strength.

In all cases photographs were taken of the specimens to show the surface texture at the top, bottom, and sides.

Tests Carried Out.

The tests that have so far been made with this machine are as follows :—

Series 1. With a constant acceleration of $4g$, the relationship between the water/cement ratio and the strength and density of the concrete has been investigated for each of four frequencies, namely, 1,500, 3,000, 5,000, and 8,000 vibrations per minute.

Series 2. With a constant acceleration of $4g$, the effect of the time of vibration on the strength and density of the concrete has been investigated for the same frequencies as in Series 1, for each of three water/cement ratios, namely 0·40, 0·50, and 0·60.

Series 3. For each of the frequencies used in Series 1, the effect on the strength and density of the concrete of varying the acceleration has been investigated for each of the water/cement ratios used in Series 2.

Results of Tests.

Series 1. In the first series a constant acceleration of $4g$ was adopted, the concrete being vibrated for 2 minutes. The effect of varying the water/cement ratio and the frequency of vibration will be considered with reference to (*a*) strength and (*b*) density. The frequencies of vibration were 1,500, 3,000, 5,000, and 8,000 per minute.

(*a*) *Strength.* The results of strength-tests are given in *Fig. 2.* From this it is seen that :—

(i) For the wetter mixes the frequency of vibration has little effect for a constant acceleration of $4g$. In *Fig. 2,* the results of strength-tests on hand-compacted concrete of the same mix have been included, and for water/cement ratios greater than 0·50 the vibrated concrete is only slightly stronger than the hand-compacted concrete.

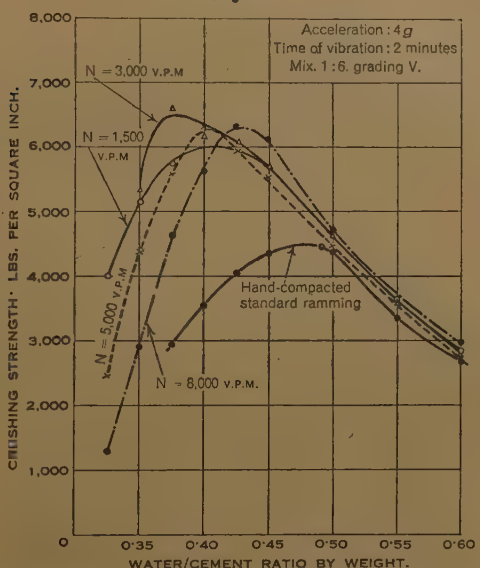
(ii) The maximum strength obtained at the critical water/cement ratio does not vary appreciably with the frequency of vibration. The variation in this maximum strength with the range of frequency in the present tests was only ± 5 per cent. from the mean value.

(iii) The critical water/cement ratio was decidedly less for the vibrated concrete than for the hand-compacted concrete, leading to an increase in maximum strength of 40 per cent.

(iv) For very dry mixes, the strengths decrease with increasing frequency of vibration. For a water/cement ratio of 0·325, the strength of the concrete made at 8,000 vibrations per minute is only one-third of that when the frequency is 1,500 vibrations per minute.

For concrete with a water/cement ratio greater than the critical value, the variability of the strength-results, expressed as the ratio

Fig. 2.

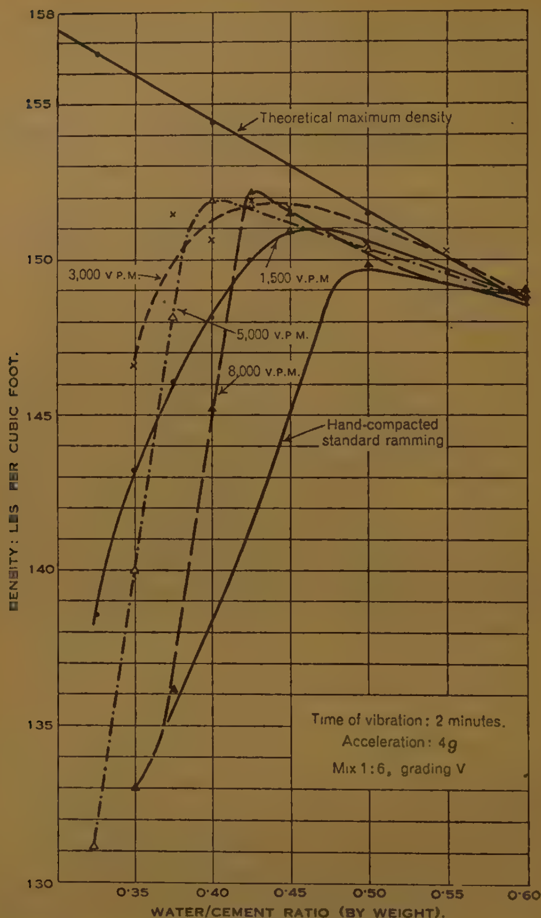


Relation Between Strength and Water/Cement Ratio for Various Frequencies.

of the standard deviation to the mean strength, was usually between 1 and 3 per cent. For concretes with lower water/cement ratios, the variability increased considerably with the dryness of the mix, and for a water/cement ratio of 0·325 the average variability was 20 per cent. There was a tendency for the variability to be greater for high-frequency vibration, particularly for the very dry mixes.

(*b*) *Density.*—The results of density measurements are expressed in this report in terms of both the actual density and also the "density-ratio," that is, the ratio of the actual density of the concrete immediately after demoulding to the theoretical maximum density,

Fig. 3 (a).



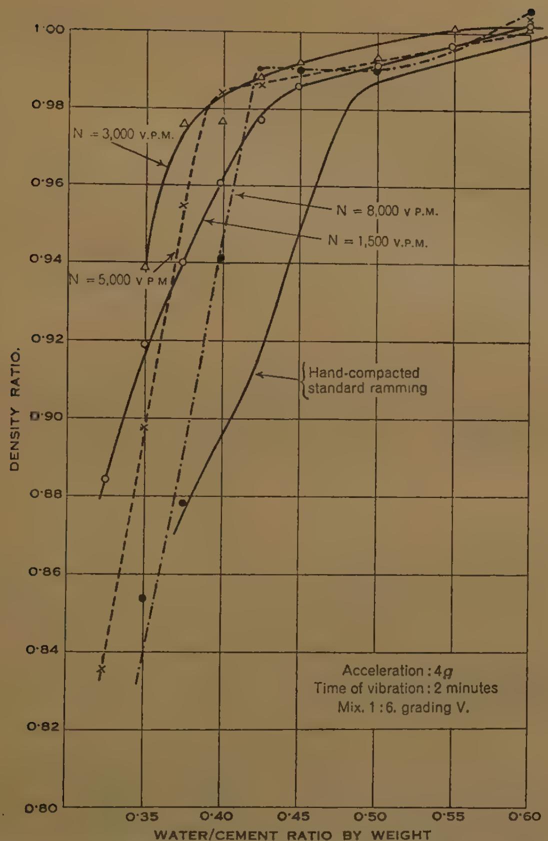
RELATION BETWEEN DENSITY AND WATER/CEMENT RATIO FOR VARIOUS FREQUENCIES.

for the particular mix, that would be obtained if there were no air voids. The results for Series 1 are shown in *Figs. 3 (a)* and *3 (b)*, and are summarized below :—

(i) For the wetter mixes the density is independent of the frequency of vibration.

(ii) Below a certain water/cement ratio the density falls off rapidly with dryness of the mix. This cricital water-content is the same as that for strength.

Fig. 3 (b).



Relation Between Density-Ratio and Water/Cement Ratio for Various Frequencies.

(iii) For water-contents less than the critical value the density is generally less for the higher frequencies of vibration.

(iv) For all water-contents, the density of the vibrated concrete is greater than that of hand-compacted concrete.

The results for the frequency of 1,500 vibrations per minute do not agree with the general trend of the results for the higher frequencies, and it appears from *Figs. 2, 3 (a)*, and *3 (b)* that the relationship between frequency and strength or density may change at a frequency between 1,500 and 3,000 vibrations per minute.

A comparison of these diagrams shows that the density-ratio and the strength-ratio (that is, the ratio of the strength to that of fully-

compacted concrete) are closely allied. In fact there was, for all Series (except for prolonged vibration beyond 12 minutes), a definite correlation between the two qualities, the relationship being dependent to some extent on the water/cement ratio. For Series 2 and 3, therefore, strength-results only are given in this interim report.

Series 2.—In the second series, the acceleration was maintained constant at $4g$ and the effect of time of vibration was determined for three water/cement ratios and for the same frequencies as in Series 1. The times of vibration tried were 5, 15, and 30 seconds, and 1, 5, and 12 minutes. A few tests have, however, been made with times of vibration of $1\frac{1}{2}$ and 3 hours. The results of the strength-tests in the main series are given in Table I.

TABLE I.—EFFECT OF TIME OF VIBRATION ON CRUSHING STRENGTH.

Water/cement ratio.	Frequency: revolutions per minute.	7-day strength of 4-inch cubes for various times of vibrations: lbs. per square inch.					
		5 seconds.	15 seconds.	30 seconds.	1 minute.	5 minutes.	12 minutes.
0·40	1,500	2,210	3,710	4,720	5,820	6,800	6,870
	3,000	2,060	3,920	4,010	5,410	6,270	6,410
	5,000	1,450	2,480	4,130	5,230	6,680	6,940
	8,000	1,210	1,910	2,660	3,180	6,270	7,110
0·50	1,500	3,730	4,210	4,230	4,200	4,310	4,150
	3,000	3,220	4,040	4,180	4,170	4,110	4,160
	5,000	2,800	4,120	4,070	3,810	4,270	4,040
	8,000	2,920	3,850	4,400	4,190	4,610	4,310
0·60	1,500	2,500	2,530	2,490	2,470	2,610	2,800
	3,000	2,270	2,350	2,570	2,390	2,630	2,835
	5,000	2,430	2,600	2,750	2,640	2,840	2,930
	8,000	2,660	2,600	2,610	2,690	2,770	2,880

The results are summarized below :—

(i) For a water/cement ratio of 0·40, the 7-day strength in general increases rapidly with the time of vibration until a critical time is reached, beyond which the strength increases more slowly. For frequencies of 1,500 to 5,000 vibrations per minute the critical time is about 3 minutes; with 1 minute vibration, the strength is about five-sixths of the value obtained with 12 minutes vibration. For a frequency of 8,000 per minute the strength for 1 minute vibration is, however, less than one-half of the value for 12 minutes vibration.

(ii) For a water/cement ratio of 0·50 ,the critical time of vibration is about 15 seconds except for the frequency of 8,000 vibrations per minute, in which case a time of 30 seconds is required to give satisfactory consolidation.

(iii) For a water/cement ratio of 0·60, the strength with only 5 seconds vibration is, for all frequencies tested, about five-sixths of the strength obtained with 12 minutes vibration.

The results for the tests with prolonged vibration are given in Table II. Only one frequency was investigated (3,000 per minute)

TABLE II.*—EFFECT OF PROLONGED VIBRATION ON CRUSHING STRENGTH.

Water/cement ratio.	Frequency: revolutions per minute.	7-day strength for 4-inch cubes for various times of vibration: lbs. per square inch.		
		2 minutes.	1½ hour.	3 hours.
0·40		6,780	7,570 (11·5)	6,740 (0)
0·50	3,000	4,660	5,380 (15·4)	5,910 (27)
0·60		2,960	4,300 (45·5)	5,040 (70)

Figures in brackets indicate percentage increase in strength over 2-minute value.

and the acceleration was again $4g$. It will be seen that for a water/cement ratio of 0·40, the strength at first increased with time of vibration and then decreased again for a 3-hour period of vibration. For both of the wetter mixes the strength increased with time of vibration, the increase being more pronounced with the mix with a water/cement ratio of 0·60. In this case, the strength obtained with 3 hours vibration was 70 per cent. greater than that obtained with 2 minutes vibration.

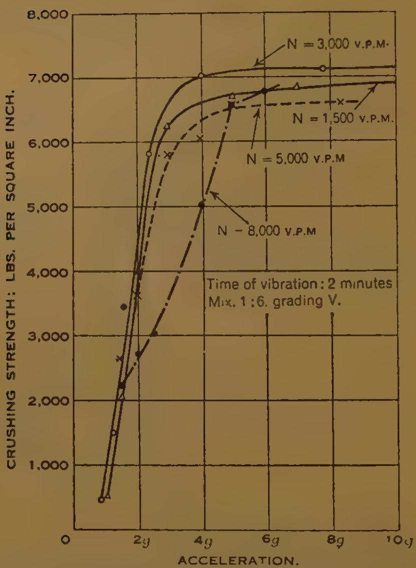
Series 3.—All tests in the previous series were made with an acceleration of $4g$, and the third series was carried out to determine the effect of variation of the acceleration. The time of vibration was kept constant at 2 minutes, and frequencies of 1,500, 3,000, 5,000, and 8,000 vibrations per minute were used for each of the water/cement ratios 0·40, 0·50, and 0·60. The results are shown in *Figs. 4, 5,* and *6* (pp. 444 and 445) and are summarized below :—

(i) In all cases there is a critical acceleration below which the strength falls off rapidly, and above which little strength-increase occurs.

(ii) For a water/cement ratio of 0·40, the critical acceleration is about $4g$ for frequencies up to 5,000 vibrations per minute. For an acceleration of $3g$ the strength is about 0·9 of that with an acceleration of $10g$; for an acceleration of $2g$ this ratio is only about 0·6. For the frequency of 8,000 vibrations per minute, the strength falls

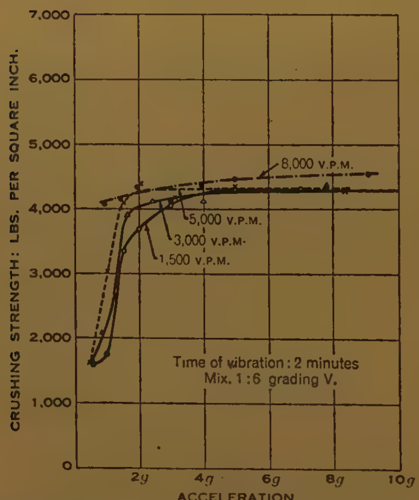
* In comparing with Table I, it should be noted that Table II refers to a different set of specimens.

Fig. 4.



Relation Between Strength and Acceleration for Various Freqencies: Water/Cement Ratio = 0·40.

Fig. 5.



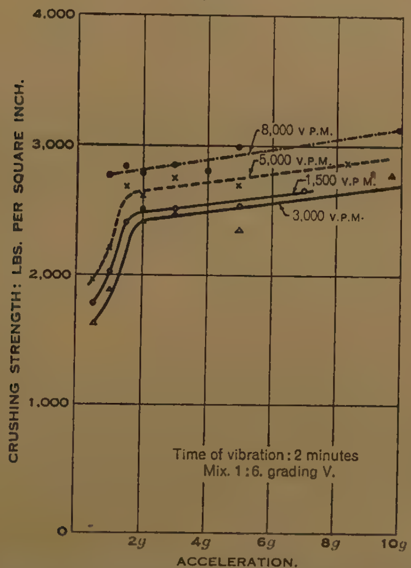
Relation Between Strength and Acceleration for Various Frequencies: Water/Cement Ratio = 0·50.

off rapidly for accelerations less than $5g$, the strength-ratio being only 0·5 for an acceleration of $3g$.

(iii) For a water/cement ratio of 0·50, the critical acceleration is about $2g$, although there is a tendency for this value to decrease with frequency from $3g$ for 1,500 per minute to $1g$ for 8,000 vibrations per minute.

(iv) For a water/cement ratio of 0·60, the critical acceleration is about $1{\cdot}5g$, although for the highest frequency an acceleration of $1g$

Fig. 6.



Relation Between Strength and Acceleration for Various Frequencies: Water/Cement Ratio = 0·60.

appears sufficient. There is a greater tendency with this mix for an increase in strength to accompany an increase in acceleration past the critical value; for an acceleration of $10g$ the strength is about 15 per cent. greater than that obtained with an acceleration of $1{\cdot}5g$.

Surface Finish.

Inspection of the surface appearance of vibrated-concrete cubes indicates that vibration at the higher frequencies gives a better surface than low-frequency vibration within the range tested. With the wet mix (water/cement ratio = 0·60) this is not very pronounced, since nearly all the air is expelled from this mix in a very short time of

vibration whatever the frequency, but even here the superiority of the higher frequencies is apparent if the acceleration is very low (below 1·5g).

For a water/cement ratio of 0·50 the tendency is clear, the airholes on a cube vibrated at 8,000 vibrations per minute being smaller and fewer in number than those on a cube vibrated at 1,500 vibrations per minute.

For very dry mixes, the results of density- and strength-tests have shown that high frequencies are not so effective for consolidating the concrete. When consolidation has been obtained, however, the surface finish obtained by the use of higher frequencies is better than for the lower values. For example, a mix of water/cement ratio 0·40 vibrated at 8,000 per minute for 1 minute is badly honeycombed, whilst a similar mix vibrated at 1,500 per minute has a fairly good surface after this time. With 5 minutes vibration, when the degree of consolidation of the concrete vibrated at 8,000 per minute is approaching that of the concrete vibrated at the lower frequency, the surface appearance is considerably better than that of the concrete vibrated at the lower frequency (even though the strength is still lower, as shown in Table I).

General Remarks on Vibration of Concrete.

In all cases the cube-mould is filled loosely by hand before vibrating. Immediately vibration occurs the mix subsides rapidly; this subsidence may be more than one-third of the total depth for a dry mix, whilst it may be only about one-fifteenth for a wet mix. After subsidence, the aggregate moves about rapidly for a few seconds and then usually settles down. In the case of a very dry mix, the top portion does not settle down if the amplitude of vibration is large (greater than about 0·08 inch) but continues to jump up and down so that a layer of stones is formed above the concrete.

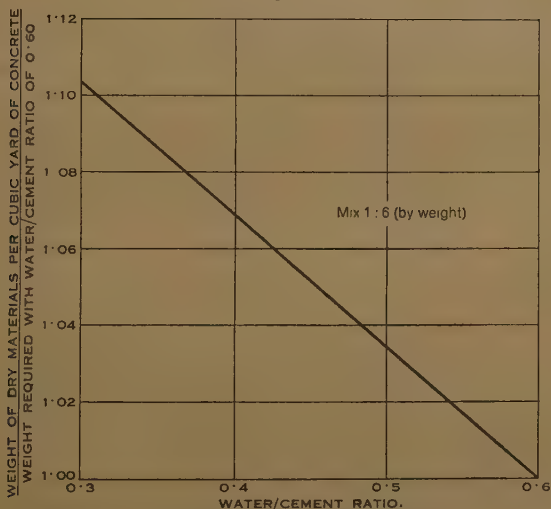
Consolidation begins at the bottom of the cube and gradually works upwards. As the solid materials become more closely packed, grout is forced outwards towards the sides of the cube and upwards into the concrete above. Until a film of grout has been formed between the bottom of the concrete and the mould, the concrete does not follow the motion of the mould but vibrates at a lower frequency, the motion being maintained by occasional impact with the mould. For water/cement ratios above 0·55 the establishment of a layer of grout is almost instantaneous. For a water/cement ratio of 0·50, it may take a few seconds, but for mixes with water/cement ratios below 0·42 the concrete may vibrate independently for a time ranging from 15 seconds when the amplitude is small to 1½ to 2 minutes when the amplitude is large. This does not mean that

consolidation does not occur during this time. Indeed, the density ratio may attain a value of 0·98 before the concrete vibrates in unison with the machine.

The rising of the grout through the concrete is a good indication of the progress of consolidation. When the grout has risen to the top in sufficient amount to enable the top to be trowelled to a fair surface it may be taken that a density-ratio of about 0·98 has been attained.

It should be realized that the use of vibratory methods of com-

Fig. 7.



THE EFFECT OF REDUCING THE WATER-CONTENT ON THE VOLUME-YIELD OF CONCRETE.

pacting, with a consequent reduction in water/cement ratio, will lead to a decrease in the volume of concrete placed for a given weight of dry materials and a particular mix.

For example, for a 1-to-6 mix (by weight), the amount of material required per cubic yard is increased by about 7 per cent. when going from hand-compacted concrete with a water/cement ratio of 0·6 to vibrated concrete with a water/cement ratio of 0·4. The increases corresponding to other water-contents are shown in *Fig. 7*.

Conclusions.

It should be clearly realized that the conclusions given in this interim report refer only to the particular concrete used in the tests and for the type of vibration described. The conclusions may require revision for larger masses of concrete, different types of vibration, or other concrete mixes.

(1) The use of vibration allows satisfactory consolidation to be obtained with a lower water/cement ratio than that used with hand-rammed concrete, with a corresponding increase of strength.

(2) In order to obtain satisfactory consolidation of concrete by vibration it is necessary that the acceleration should be above a certain critical value. For the particular mix used, this value is about $4g$ for a water/cement ratio of 0·40, decreasing to $1{\cdot}5g$ for a water/cement ratio of 0·60, when the time of vibration is 2 minutes.

(3) The frequency of vibration is not normally of such importance as the acceleration. For very dry mixes a low frequency is better than a high frequency as regards strength and density of the concrete. For wet mixes a high frequency is more useful. For all water-contents, the use of a high frequency will lead to a better surface finish, so long as the vibration is sufficient to consolidate the concrete.

(4) The time of vibration required to consolidate concrete is, for a particular acceleration, greater for higher frequencies, particularly with dry mixes. The increase of strength with time of vibration is relatively slow after a critical time is exceeded. This time, for an acceleration of $4g$ and the particular mix used, ranged from about 5 minutes for a dry mix with a frequency of 8,000 vibrations per minute to 5 seconds for a wet mix for all frequencies.

When the time of vibration is considerably extended, the strength at 7 days may be increased appreciably, particularly with wet mixes. With a water/cement ratio of 0·60 the strength for 3 hours vibration was 70 per cent. greater than for 2 minutes vibration.

In general, the results of the tests with the large machine are in agreement with, and supplement, the tentative conclusions drawn from the earlier tests on the small machine.

On p. 318 of this Journal is given a Paper by Mr. D. S. Stewart on "Fundamental Research on the Application of Vibration to the Precasting of Concrete." Mr. Stewart has investigated the problem by means of an electrical machine in which it has been possible to study the change in energy consumed during the process of compaction. It may be of interest to note the points of difference between his experiments and results and those of the Building Research Station. In the first place the Building Research Station, for their experiments on the large machine, used aggregate of $\frac{3}{4}$-inch maximum size, in contrast to Mr. Stewart's $\frac{3}{8}$-inch maximum size. The proportion of sand in the tests at the Station was 30 per cent. by weight of the total aggregate (that is, 43 per cent. of the coarse aggregate), whereas in Mr. Stewart's tests the proportion was 38 per cent. of the total aggregate (that is, 60 per cent. of the coarse aggregate). Early tests at the station had shown that for optimum

results the proportion of sand should be reduced for the drier mixes possible with vibrated concrete. Mr. Stewart's experiments appeared to indicate that, with a given acceleration, a high frequency reduced the energy consumed per lb. of concrete consolidated, but that further investigation was necessary.

Further experimental work is indicated to investigate, *inter alia*, the relative efficiency of various types of vibration; for example, simple-harmonic vertical vibration, horizontal vibration and vertical circular vibration; the effect of the maximum size of aggregate, the optimum grading, the effect of the type of aggregate, the type of cement, etc. In addition there remain to be carried out full-scale tests of commercial vibrators as applied to concrete in situ.

APPENDIX.

Definitions and Relationship between Frequency, Amplitude, and Acceleration.

Definitions.

The Frequency of vibration is the number of complete cycles per unit time, and in this report unit time is taken as 1 minute.

The Amplitude of a vibrating body is the maximum displacement of the body from its mean position.

Acceleration, in this report, refers to the maximum acceleration; that is, the maximum rate of change of speed of the vibrating body.

If the frequency is N vibrations per minute and the amplitude is a inches, the acceleration f in terms of the normal acceleration g due to gravity is given by

$$f = \left(\frac{2\pi N}{60}\right)^2 \times \frac{a}{12} \times \frac{1}{32 \cdot 2} \times g = \frac{N^2 ag}{35{,}235}.$$

Table III.—Amplitudes for Various Frequencies and Accelerations.

Frequency: Vibrations per minute.	Amplitudes for accelerations of:			
	1g	2g	4g	10g
	inch	inch	inch	inch
1,500	0·0157	0·0313	0·0627	0·1566
3,000	0·0039	0·0078	0·0157	0·0392
5,000	0·0014	0·0028	0·0056	0·0141
8,000	0·0006	0·0011	0·0022	0·0055

RESEARCH WORK IN ENGINEERING AT VICTORIA UNIVERSITY, MANCHESTER, FEBRUARY, 1937.

Engineering research is being actively pursued in the Departments of Engineering and Electrotechnics at Victoria University.

Engineering Department.

Many classical researches in hydraulics have been carried out in the Engineering Department. In recent years much work in the development of the theory of dynamic similarity has been done and the technique of investigation by hydraulic models has been developed. Mention may be made of the Severn Barrage investigation, in which it was possible to compare changes in the bed of the estuary and the model over a period of 80 years. A further research with a model of the Severn estuary which has just been completed and which is the subject of a Paper [1] recently presented to The Institution, shows the effect on tide- and river-levels of varying degrees of obstruction such as would be produced by bridge piers. Some rather unexpected results have been noted. An investigation of the estuary of the Dee to determine the probable effect of bridge piers and training works by means of a tidal model is in preparation. With a view to preventing flooding and to the cutting down of expenditure on maintenance, a study is being made by means of a scale model of the effect of shortening and straightening the river Mersey by the construction of certain cut-offs. A model-study is also being made of the transportation of detritus in a flowing stream. Model-investigations are in progress into the flow over weirs and into the variation of the coefficient of discharge with the form of cross-section.

In the realm of structural engineering, an experimental and theoretical investigation is being carried out into the stresses in grain-bins, and it has been possible to explain some hitherto unexplained failures in large grain-bins. A research is being conducted into the torsional properties of the British Standard beam-sections.

On the mechanical engineering side, researches have been instituted into problems connected with the operation of the internal-combustion engine. An investigation extending over several years has been carried out into the heat-transference to the cylinder walls,

[1] A. H. Gibson, "An Experimental Investigation of the Effect of Bridge-Piers and other Obstructions on the Tidal Levels in an Estuary." (This Paper will be published later in the Institution Journal.—Sec. Inst. C.E.)

piston and exhaust valves, and at the present time a study is being made of piston temperatures in high-speed compression-ignition engines. A further research in progress is a study of the pressure-oscillations in the exhaust-pipe of a high-speed engine with a view to the possibility of improving the performance by modification in the design of the exhaust-pipe.

Mention may also be made of recent work on the stresses produced by surges in electrical transformers, and an investigation of the piezometric effect in quartz crystals, with a view to measuring cavitation pressures caused by collapsing bubbles in regions of low pressure in pumps and turbines, has been initiated.

Department of Electrotechnics.

Electrical engineering research has developed mainly along the lines of high-frequency work and investigations in this direction are in active progress. Recent work includes investigations on diode rectifiers, measurement of current and voltage at radio frequency, stability of radio-frequency amplifiers, operation of frequency-changing valves, methods of calibrating cathode-ray oscillographs, simultaneous recording on cathode-ray oscillographs, the design of wide-band amplifiers, and experiments on television. In addition, research has been carried out on the mercury-arc rectifier.

The above researches are being carried out under the direction of Professor A. H. Gibson, D.Sc., LL.D., Professor of Engineering, and Professor Robert Beattie, D.Sc., Professor of Electrotechnics.

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH.

Report for the year 1935–36.

The Annual Report of the Department (just issued) indicates an encouraging trend in industry to take up new ideas and to bring them to the stage of industrial application by team work in which the scientist, the technical expert, and, in fact, all the departments into which a great business is organized, have worked side by side in the practical attainment of an objective. Great importance is attached to the development of the Co-operative Research Associations. The income subscribed by industry has increased during the last 3 years from £167,370 to £232,468, while the grants from the Department have increased from £68,212 to £107,451. While the year under review has shown the greatest increase in research activity to date, it is pointed out that the progress is not yet entirely satisfactory.

The Department is at present, and has been for the last 2 years, prepared to provide a further £66,000 each year. This would have represented an increase of income to the Research Associations of £150,000 a year if full advantage had been taken of this offer by industry.

During the year new laboratories have been opened for the Electrical Research Association at Perivale at a cost of over £29,000, and for the Institution of Automobile Engineers on the Great West Road at a cost of £20,000. The laboratories of the Paint Research Association and of the Cotton Research Association have been extended, the latter at a cost of £41,000.

The Report covers a wide field of engineering and other scientific research. A certain amount of this has already received brief mention in the Institution Journal under the yearly reports of various research boards. In the June, 1936, Journal appeared a note on the report of the National Physical Laboratory,[1] an account of the work being carried out by the Automobile Research Association,[2] and a Paper by Professor J. F. Baker summarizing the Final Report of the Steel Structures Research Committee.[3] The report on Forest Products Research [4] was dealt with in the November, 1936, Journal, that of the Fuel Research Board [5] in the December, 1936, Journal, and that of the Building Research Board [6] in the Journal of February, 1937.

During the year the Illumination Research Committee has been constituted a standing committee of the Department. A committee has been formed to investigate gas cylinders and containers. Of particular interest to engineers in the report of the Geological Survey and Museum of Practical Geology are the investigations into the continuous fall of water-table in the chalk under London, and exploratory work in connexion with boring for petroleum.

The report of the Road Research Board describes methods for grading aggregates according to shape; mechanical tests for bituminous materials, research into road tar, control of water-content in concrete mixes and tests for workability, the design of concrete road-slabs and tests of various road-surfacings.

On the subject of Metallurgy further research has been carried out on the behaviour of metals under deformation at high temperatures, the effect of age-hardening on the resistance to creep of nickel-silicon-copper alloy, the influence of initial conditions and the

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1 Journal Inst. C.E., vol. 3 (1935–36), p. 343.
2 *Ibid.*, p. 345.
3 "The Rational Design of Steel Building Frames." *Ibid.*, p. 127.
4 *Ibid.*, vol. 4 (1936–37), p. 153.
5 *Ibid.*, p. 315.
6 *Ibid.*, vol. 5 (1936–37), p. 230.

manufacturing processes of steels on their properties at high temperatures, the behaviour of cast irons at elevated temperatures, the creep of aluminium alloys and magnesium alloys, the nature of creep under complex stress-systems, and exploratory work to discover new types of materials suitable for high-temperature-work. Research is also being carried out on light alloys, alloy steels, cracking of boiler-plates, the effect of gases in steels, the control of grain-size, the estimation of oxides in steel, and the oxidizing power of basic slabs.

In connexion with Water Pollution research has been carried out on the base-exchange process of water-softening, the exchange properties of synthetic resins, the contamination of water by lead, milk-factory effluents, and the purification of sewage, and an investigation has been made of the pollution of the river Mersey.

The report on Chemical Research contains much matter of engineering interest such as the work on corrosion of metals, chemical reactions at high pressure, tar research, rubber research, and work in connexion with water pollution.

The Report also contains sections dealing with Radio Research, Illumination Research, Lubrication Research, Furnace Design, and X-Ray Analysis of Materials. A summary is given of the work of the Research Associations on cast iron, iron and steel, non-ferrous metals, refractories, electrical research, scientific instruments, paint, automobile engineering, rubber, and mining. In addition, a large amount of research of a non-engineering nature has been carried out.

NOTES ON RESEARCH PUBLICATIONS.[1]

Measurement.

A manual: The theory of dimensions and its application for engineers, by F. W. Lanchester, London, 1936, has been published. In the *Journal of the Engineering Association of Malaya*, **41,** *300*, the measurement of the discharge of sharp-edged weirs with free nappes is discussed.

Engineering Materials: Properties and Testing.

The permeability of structural elements, building materials, and heat-insulating materials to air, as measured at the Research Laboratory for Heat Insulation, Munich, is dealt with in *Ver. deutsch. Ing.*, **80,** *125*.

[1] The figure in heavy type is the number of the Volume; that in brackets the number of the Part; and that in italic type the number of the Page.

Timber.

The reduction in strength of wood beams as the result of notches is shown in *Ver. deutsch. Ing.*, **80,** *470,* to depend more on the depth and radius of the notch than on its shape and width.

Bricks, Cement, and Concrete.

The thermal behaviour of bricks and its determination are described in *Rev. Mat. Constr. 1936* (*325*), *145–8B;* (*326*), *165–9B.* The addition of powdered stone to the cement is shown to cause only a slight reduction of strength in mortar test-cubes, but the strength of concrete is shown in *Science et Ind.* (*Travaux*), **20,** *385,* to be in accordance with the reduced cement-content of the mix. The effect of departures from planeness in the surfaces of the bearing blocks on the compressive strength of mortar cubes is discussed in *U.S. Nat. Bur. Stds. Tech. News Bull., 1936* (*231*) *61.* In *Eng. News-Record,* **117,** *541,* the ratio between the compressive and the flexural strength of concrete is shown to be dependent upon the water/cement ratio. A paper on the deformation of concrete under load is published in *Municipal Engineering,* **48,** *626.* Experiments on the leaching-out of lime from concrete by the action of distilled water are given in *Bautenschutz,* **7,** *105.*

Metals.

The Presidential Address of Sir William Larke, on metallurgical development and engineering progress, is given in *J. Junior Inst. Eng.,* **47,** *125.* The plastic flow of metals is dealt with in *Virginia Poly. Inst. Bull. No. 27.* The raising of the yield-point in tension and bending under conditions of irregular stress-distribution are discussed in *Bauing,* **17,** *431.* The mechanical properties of manganese-copper steel are dealt with in *Stroitelnaya Promishlennost,* **14,** *27,* and the age-hardening of aluminium-silicon alloys in *Zeit. Metall.,* **29,** *25.* An account of experiments on the atmospheric corrosion of steel under varying conditions of rainfall, humidity, and temperature is given in *Zeitschrift für Elektrochemie,* **42,** *107,* and a discussion of the desirability of standardizing a corrosion test is given in *Zeitschrift für Metallkunde,* **28,** *25.*

Engineering Materials: Production, Manufacture, and Preservation.

Cement and Concrete.

A study to determine the most favourable curing-conditions for concrete is described in *Rev. Mat. Constr., 1936* (*322*), *152.* The

production of light-weight concrete by the use of various foaming agents, and its stability during setting, are discussed in *Zement*, **25,** *882*. The composition of cements resistant to the corrosive action of sodium sulphate solutions is given in *J. Am. Concr. Inst.*, **8,** *83*.

Metals.

Consideration is given in *Ossature Métallique*, **5,** *516*, to the protection of underground steel pipe-lines against stray tramway currents by the introduction of direct electrical connexions. The production of black anodic coatings on tin and tin alloys is the subject of *Tech. Pub. International Tin Research and Development Council*, Series A, *No. 48*.

Structures.

Mass Structures.

Methods based on the variation in the electrical resistance of different soils and on seismic tests for determining the most suitable position for the foundations of a structure are given in *International Association for Bridge and Structural Engineering, Publications*, **4,** *522*, and on *p. 279* of the same publication the design of reinforced-concrete foundation slabs is discussed. A graphical method for determining the distribution of vertical pressures beneath foundations is given in *Proc. Am. Soc. C.E.*, **63,** *3*. A rational method of design of pile foundations is given in *Tech. Memo. No. 531, U.S. Bur. of Reclamation.* A large-scale experiment on a method of increasing the carrying capacity of floating pile-foundations by electro-chemical treatment is described in *Bautech*, **15,** *14*. The stability of rock-fill weirs on pervious foundations is dealt with in *Tech. Memo. No. 490, U.S. Bur. of Reclamation*, and in *No. 533* a description of laboratory procedure in testing earth-dam materials is given. The effect of an end line of sheet-piling on the pressure-distribution under a weir floor and on the exit gradient is dealt with in *Punjab Irrigation Research Inst., Res. Pub.*, **2,** *14*, and the determination by electrical and by graphical methods of the uplift pressures under weirs with three lines of sheet-piles in *Proc. Indian Acad. Sciences*, **4,** *491*. The stresses in dams are dealt with in the following *Technical Memoranda of the U.S. Bureau of Reclamation: No. 424*, Comparison of the measured and calculated radial displacements near a cooling pipe embedded in concrete: *No. 539*, Analysis of the arches of a multiple-arch dam.

Framed Structures.

Consideration is given to errors in the methods of calculation concerning the strength of materials and the theory of elasticity in

International Association for Bridge and Structural Engineering, Publications, 1936, **4**, *411*, and on *p. 543* are given the results of experimental research on the flow of stresses in a beam, with special reference to the high stresses near point loads. In *Univ. Iowa Engineering Expt. Stn. Bulletin 129*, a theory of thin rectangular plates supported on opposite edges is given, with experimental verification. A method of geometrical analysis for statically indeterminate structures based upon principles pertaining to unloaded models is given in *Proc. Am. Soc. C.E.*, **63**, *15*. Stresses in thin conical shells of constant thickness are discussed in *Technical Memo. No. 534, U.S. Bureau of Reclamation.* The theory and calculation of self-supporting steel shells is considered in *International Assoc. for Bridge and Structural Engineering, Publications, 1936*, **4**, *155*; on *p. 131* is given an exact theory of thick cylindrical shells based upon the deformations and equilibrium conditions of an infinitely small particle; and on *p. 469* the calculation of the two-hinged ribbed dome with an elastic ring at the base is discussed. The report of the Steel Structures Research Committee is outlined on p. *113*; on *p. 217* appears a treatise on a particular type of frame construction in which the horizontal thrust may be reduced to zero; on *p. 227* the theory of the continuous stiffened cylindrical tube, taking into account deformation due to shear and its application to the Zeiss-Dywidag roof, is considered; and on *p. 429* the economy resulting from the design of arched bridges with inclined hangers is explained. Coefficients of friction of sliding bearings for bridges are given in *Public Roads*, **17**, *223*. Tests on overstrained reinforced concrete are detailed in *Stroindustria*, **4** (*11*) *73*. The strengths of plate girders of timber with plywood webs connected by various methods are compared in *Eng. News-Record*, **117**, *534*. A book entitled "Wind-pressure on Buildings. Experimental Researches (Second series)," by J. O. V. Irminger and C. Nøkkentved, a translation from the Danish, describes research at the Royal Technical College, Copenhagen. The effects of the Quetta earthquake on various types of construction are analysed in *J. Royal Engineers, 1936*, *50* (*September*), *327*.

Constructional Operations and Methods.

A method of improving the strength and density of concrete of soft consistency by the use of special suction mats through which the excess water is drawn is described in *Teknisk Tidskrift*, (*Allmänna Avd.*) **66**, *557*. A method of concreting the full circle of a tunnel-lining in a single pour is described in *Eng. News-Record*, **117**, *743*.

Transformation, Transmission, and Distribution of Energy.

In *J. Inst. Fuel*, **10,** *68*, some measurements by gas analysis of the efficiency of the locomotive furnace are given, and on *p. 79*, combustion control by means of electrical meters is discussed. A new design of electric boiler is described in *Brown-Boveri Review*, **23,** *263*. Data regarding the wear of piston-rings in crude-oil engines are given in *Stahl und Eis.*, **57,** *42*. A description is given of pressure-relieved packing-rings for steam and gas pistons in *Wärme*, **60,** *38*.

The following electrical researches have been noted: Ironless cylindrical coils with non-uniform winding-depths for the production of uniform fields, *Archiv Elek.*, **30,** *736*; in the same journal, *p. 677*, Rupture and dielectric stresses of insulating oils of different dielectric strengths; *p. 651*, Exact measurement of the electrical conductivity and permeability of iron wires; *p. 637*, Transient induced voltages in single-earthed conductors; in *J. Inst. Elec. Engineers*, **80,** *80*, Pollution of suspension-type insulation on overhead lines; *p. 99*, Mutual impedance of circuits with return in a horizontally-stratified earth; *p. 218*, The response of rectifiers to fluctuation voltages; and in *Rev. Gén. Elec.*, **41,** *13*, Corrosion of 220-kv. line conductors by an electrolytic effect.

Mechanical Processes, Appliances, and Apparatus.

The results of an experimental determination of the distribution of stresses in fillet welds are discussed in *J. Inst. Eng. Australia*, **8,** *286*. In *Welding Industry*, **5,** (*1*) *8*, new techniques for oxy-acetylene butt welding of steel are considered, and on *p. 24* are given working-data on the cutting of steel by the oxygen-jet.

Specialized Engineering Practice.

Transport.

A paper on soil stabilization in highway construction is given in *Canadian Engineer*, **71,** *43*. The structural design of concrete pavements is dealt with in *Public Roads*, **17,** *143*. A note on vibrated concrete, with special reference to its use in road construction, giving the results of an investigation at the Research Institution for Road Construction, Berlin, is given in *Zement*, **25,** *907*. A publication, "Pressure-distribution on the contact surface between tire and road," by H. Martin, Berlin, 1936 (*V.D.I.-Verlag*) gives a simple theory for the distribution of pressure-intensity with solid and pneumatic tires. The following articles have been noted in *J. Inst. Auto. Engineers*, **5**: *No. 3, p. 20*, Brake-performance, *p. 62*,

Processes in oil-engine injection systems with spring-loaded nozzle-valves, and *p. 86*, Compression-swirl oil engines; *No. 4, p. 20*, The four-cylinder engine—maximum performance, life, and refinement; *No. 5, p. 17*, Units and dimensions—the treatment of problems in engineering by dimensional theory. Consideration is given to the buckling of railway-tracks without joints in *Rev. Gén. Ch. de F.*, **55**-*ii*, *212*. The corrosion of steel railway-tracks by coal is discussed in *Railway Age*, **101**, *780*.

Research in connexion with water transport includes a paper on designing for economy in ship propulsion, *J. Inst. Marine Engineers*, **49**, *1*. The influence of surface roughness upon ship performance is discussed in *Werft*, **18**, *19*. The *Journal* of the *Association Technique Maritime et Aéronautique*, **40**, contains articles on The constitution and efficiency of anti-fouling compositions, *p. 239*, and Cavitation phenomena in propellers, *p. 701*.

Researches in connexion with air transport include: Measurement of circulation along the span of a wing, *Comptes Rendus*, **204**, *104*; a paper in *J. Roy. Aer. Soc.*, **41**, *68*, on the part played by skin-friction in aeronautics. The following Aeronautical Research Committee Reports and Memoranda have been noted; *No. 1701*, The effect of central cutaway in split flaps on the trim of a low-wing monoplane; *No. 1706*, Tests of six aerofoil sections at various Reynolds numbers in the compressed-air tunnel; *No. 1707*, Trailing-edge flaps in relation to take-off and landing of landplanes; *No. 1708*, Effect of surface roughness on characteristics of aerofoils N.A.C.A. 0012 and R.A.F. 34; *No. 1714*, Interim report on systematic model research in free spins; low-wing monoplanes; *No. 1715*, A new form of biplane; *No. 1718*, The use of dynamically similar models for determining the porpoising characteristics of seaplanes; *No. 1719*, Full-scale tests of the Hendy Heck; *No. 1720*, Preliminary calibration of the 24-foot wind tunnel, R.A.E., with a short description of the tunnel; *No. 1721*, The pressure-distribution and forces on thin aerofoil sections having sharp leading and trailing edges, and moving with speeds greater than that of sound; *No. 1722*, The effect of a reduction of aileron torsional stiffness on the flutter of a model wing; *No. 1724*, Full-scale and model resistances of a Southampton II hull; *No. 1725*, Turbulence measurements in flight; *No. 1728*, Note on wind-tunnel tests on a Parasol monoplane with Zap and split flaps; *No. 1732*, Note on effects of landing flaps on stability and control; *No. 1737*, Notes on the optimum stiffness of thin shells; *No. 1738*, Boundary-layer growth; *No. 1739*, Note on the velocity-distribution in the wake behind a flat plate placed along the stream; *No. 1743*, Rolling experiments on a " Puss Moth " Model.

Water-Supply and Sewage-Disposal.

An electrical method for the measurement of the flow of subsoil water is given in *De Ingenieur* (*Bouw- en Waterbouwkunde*), **52,** *17*. In *Purdue Univ. Eng. Bull.*, **20,** *1* (*Research Series No. 53*), the electrical conductivity of fire streams, a research to find a safe distance at which a fireman might direct a hose stream on high voltage lines, is described.

A progress report of the Committee of the Sanitary Engineering Division on standard practice in separate sludge-digestion is given in *Proc. Am. Soc. C.E.*, **63,** *39*, and in *Sewage Works Journal*, **8,** *904*, an experimental high-rate trickling filter for sewage is described; on *p. 915* the detention of liquids in continuous-flow tanks is discussed, and on *p. 924* a comparison is made of compressed-air and mechanical aeration of activated sludge.

Mining.

Illumination contours for miners' lamps and underground illumination surveys are discussed in *Trans. Inst. Mining Engineers*, **92,** *107*, and on *p. 158* is given a report of the Safe Working of Mines Committee on the support of underground roads. The influence of the speed of detonation of an explosive on the speed of the shock-wave is considered in *Comptes Rendus*, **204,** *179*. A paper on wire-rope problems is given in *Proc. S. Wales Inst. Engineers*, **52,** *327*. Some chemical and physical considerations involved in the cementing of oil-wells are given in *J. Inst. Petroleum Technologists*, **22,** *798*, and in the same journal, **23,** *1*, information is given concerning temperature measurements in oil-wells.

Heating, Ventilation, and Acoustics.

A paper on the heat-requirements of a house, based on research at the Building Research Station, is given in *J. Inst. Heat. and Vent. Engineers*, **4,** *560*. The heat-insulation of building materials according to the results of the most recent research is discussed in *Schweizerische Blätter für Heizung und Lüftung*, *1936* (*4*), *73*.

A paper on ventilation with air conditioning in modern buildings is given in *J. Inst. Mech. Engineers*, *February*, *1937*, *p. 43*.

The cause and effect of noise are considered in a book entitled "Noise," by H. Wigge, Leipzig, 1936. An investigation of the relation between impact noise and floor construction is described in *Builder*, **151,** *484*.

Telegraphy and Telephony.

The following researches have been noted: The damping and propagation-time in cables with wide frequency-bands, *Archiv Elek.*, **30**, *691*; In *Hochfrequenztechnik*, **48**, *p. 182*, Coupling impedance of coaxal conductors, *p. 191*, Disturbances in broad-band cables due to high-frequency transmitters, *p. 201*, Recent investigations of decimetre-wave transmitters with split-anode magnetrons; and in *J. Inst. Elec. Engineers*, **80**, *84*, Electronic oscillations in positive-grid triodes and resonance oscillations in magnetron generators.

MISCELLANEOUS.

A report of the research and extension activities of the Purdue University is given in *Purdue Univ. Eng. Bull.*, **20**, *5* (*Research Series No. 55*). In *U.S. Nat. Bur. Stand. Hydraulic Laboratory Bull. V-1*, the current hydraulic laboratory research in the United States is reviewed.

NOTE.

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